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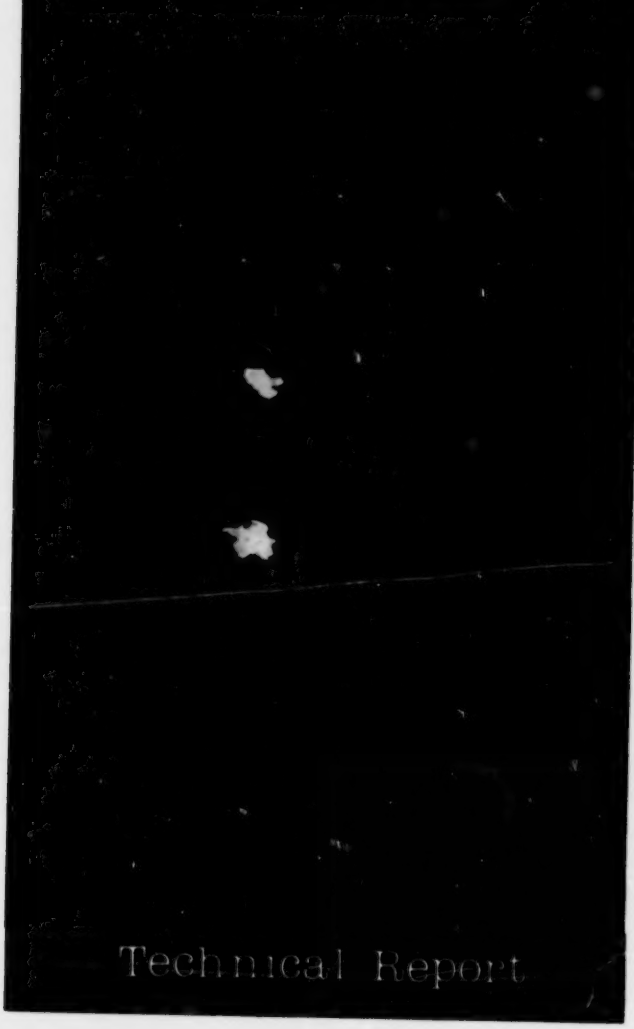

Test Platforms for the Measurement of Ice Forces

edited by: Alastair J. Allan
David P. Bazeley

May 1981

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Technical Report

C-CORE Publication No. 81-6

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CENTRE FOR COLD OCEAN RESOURCES ENGINEERING
MEMORIAL UNIVERSITY OF NEWFOUNDLAND
ST. JOHN'S, NEWFOUNDLAND, A1B3X5, CANADA
TELEPHONE [redacted] (709) 737-8354

The Centre for Cold Ocean Resources Engineering (C-CORE) was established at Memorial University of Newfoundland in St. John's in 1975. It's initial funding was provided by the Devonian Group of Charitable Foundations of Calgary. This support is being supplemented by grants from governments and industry.

The Centre's mandate is to undertake research that will assist the safe and orderly development of Canada's resources, in which cold and ice frequented oceans are a problem. C-CORE's main concentration of research is on ice, its properties, detection and behaviour.

Alastair J. Allan
Assistant Director
C-CORE

David P. Bazeley
Leader, Publications Group
C-CORE

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PREFACE

In the spring of 1979, C-CORE researchers occupied a steep, rocky islet, Gull Island, off the north east coast of Newfoundland, to assess its suitability as a test "structure" for the analysis of ice/structure interaction. The project was primarily concerned with tracking ice movement and observing its mode of failure, pile up and flow around the island. The results were so encouraging that we were led to consider the possibility of measuring in full scale the forces exerted on the test island due to ice impact. This work is the result of our subsequent deliberations. It is not intended to be a definitive study of the measurement of ice forces, but more a collection of thoughts to stimulate further field investigations on the use of islands as test structures.

Four topics are considered:

1) A brief overview by Dan Masterson of the large volume of available data literature on the topic of ice forces and structures;

2) An examination of the rationale and methodology of ice force measurement systems by John Jones;

3) The identification of a list of possible island sites by Chantal Roche. The search was confined to suitable islands likely to be impacted by multi-year ice;

4) An appraisal by D.V. Reddy and Asim Haldar of an alternative concept, that of a specially designed cold ocean test structure.

We are very grateful to the authors for their contribution of this volume, to Mike Booton of Memorial University's Faculty of Engineering and Tom Laidley of C-CORE for their helpful and constructive comments, and to Cameron O'Rourke of Dome Petroleum Ltd. who first suggested the undertaking and providing the initial funding.

Alastair Allan

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A REVIEW OF ICE FORCES ON OFFSHORE STRUCTURES

by

D.M. Masterson
FENCO Consultants Ltd.

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1. Introduction

The potential reserves of hydrocarbon deposits in offshore areas of the Canadian Arctic are regarded as very large. Numerous exploratory wells have been drilled from drillships, artificial islands and ice platforms with promising results. A great deal of effort is now going towards the design of methods to produce petroleum from these offshore areas. One promising method for producing in deep water (over 20 m) is the use of a fixed structure holding production equipment above the water level. This type of platform is now used in Sub-Arctic and temperate regions throughout the world but new designs and concepts are required in Arctic areas due to the very large forces exerted by floating ice on such structures. The forces imposed by ice are much larger than any other environmental force.

The study of ice forces on structures has been active for many years and is accelerating rapidly. Numerous conferences have been held around the world dealing with the topic and the literature is voluminous. This report is a brief review of the literature as it applies to an offshore platform in the Canadian Arctic and particularly to the development of a design for a full scale test structure. The review is necessarily brief but is intended to give a basic knowledge of the state of the art. Our knowledge of ice failure mechanisms and resulting forces on structures has advanced rapidly over the past few years and is continuing to grow.

2. Forces Exerted by Ice Sheets on Vertical Columns

The vertical column is the simplest structure for ice force analysis and consequently is the most commonly examined case. The circular column is the basis of many offshore structures (e.g. wharves, jack platforms, monopods, etc.) and will be considered here. The ice forces on such a column are associated with the crushing failure of the ice as the ice sheet, floe or fragment moves past the column. The force of the ice is often parameterized by the "effective pressure" P_e which is the average pressure of the ice over the area of contact.

$$P_e = F/dh$$

where:

F = total force on column

d = diameter of column

h = thickness of ice sheet

Most authors analyzing the force imposed by ice sheets on vertical columns relate the effective pressure on the column to the crushing strength of the ice (Korzhavn, 1962, Michel, 1970, Danys, 1975, etc.). Other factors in the determination of effective pressure may include shape coefficients, contact coefficients, velocity terms, and indentation coefficients. The indentation coefficient C_i is included to account for the aspect ratio or ratio of ice floe width to column diameter. Ignoring other factors, the indentation coefficient is the ratio of the effective pressure on a structure to the compressive strength of the ice.

$$C_i = P_e/\sigma_c$$

where:

σ_c = crushing strength of ice

The dependence of the indentation coefficient on the aspect ratio is shown in Figure 1, as determined by Afanasev (1971) and as measured by various experiments. It can be seen that for column diameters greater than twice the ice thickness any changes in column size have small effects.

Theory and experiment have shown that effective pressure in a crushing failure should depend on the ratio of ice floe thickness to column diameter, the shape of the column, ice strain rate, floe velocity, material properties of the ice and temperature (Neill, 1976). At present it does not seem possible to give an exact analytical solution including all variables; hence an empirical value of P_e is often designated in guidelines and used for initial designs. Such guidelines are commonly used for highway bridges and other river structures, with recommended values for P_e varying from 295 kPa in U.S.S.R. to 2760 kPa in Canada (Iyer, 1978).

There have been many attempts to devise analytical or empirical formulae which can predict effective pressures over a wide range of conditions but still many questions are unanswered concerning the failure of ice sheets and the pressures exerted by inhomogeneous ice in moving floes of diverse shapes and sizes. The effects of velocity are not agreed upon (Korzhavn, 1962, Frederking and Gold, 1971) and temperature effects are not well known (Blenkarn, 1970). There is at present no universal or common method of testing ice and few connections between different experiments, test results and theories.

The dynamics of ice-structure interaction have been considered by numerous authors but there is not yet a theory which completely explains the vibratory forces measured on platforms in Cook Inlet (Blenkarn, 1970 and Peyton, 1968). Force oscillations of low and high frequency have been described (Kivisild, 1968) with reference to physical processes in the ice and dynamic response of the structure. Further work is required in the dynamics of structures impacted by large masses of floating ice and

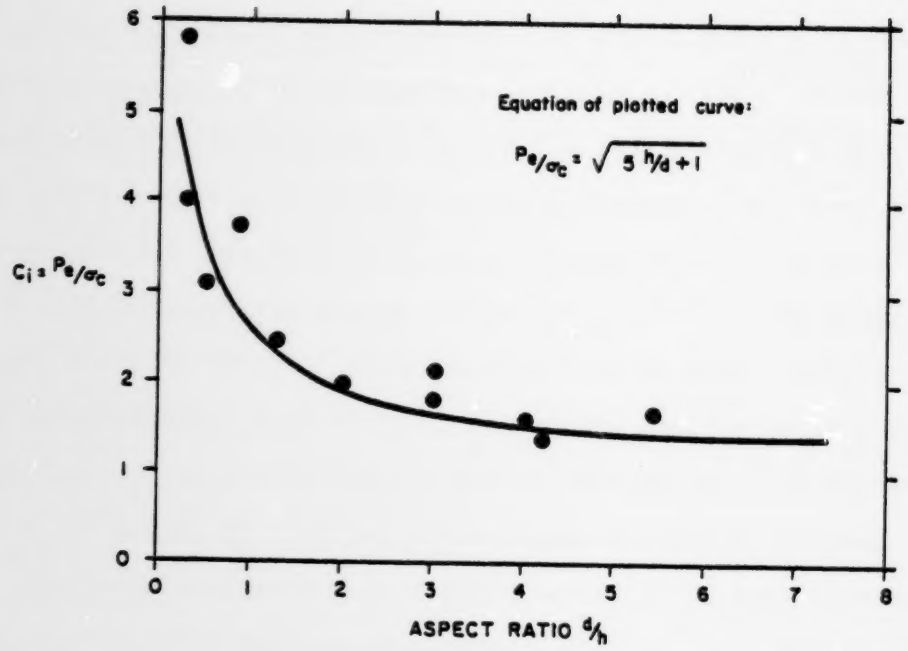

Figure 1 - Coefficient of Identation vs Aspect Ratio (Afanasev, 1971)

by fields of unconsolidated ice floes.

The possibility exists that an ice sheet in compression may buckle rather than fail in compression. The minimum sheet force is taken when designing. The critical buckling stress corresponds very closely to the minimum energy to failure of the ice sheet and these stresses determine the general pack strength. For impact against relatively slender columns, the stress at the column can be higher than these minimums because of local effects (Kivisild, 1976, and FENCO, unpublished reports). More will be said about pack strength when doing force calculations on a monocone.

3. Forces Exerted by Ice Sheets on Cones

The forces of ice impacting a sloping structure may be substantially lower than for a vertical structure if the ice fails in flexure rather than compression. Bending strengths of ice are on the order of 1/2 to 1/3 of compressive strengths with similar reductions of total horizontal force on the structure. Kivisild (1975) describes methods of testing the various ice strengths and shows the material properties to be considered. Structures encountering ice in rivers are usually designed with a sloping face and considerable analysis of such interaction is found in the literature (Watts and Podolny, 1976, Neill, 1976). In most offshore sites ice impact may occur on any side; hence a conical structure is required to present a sloping face to all impacts. This shape poses a more difficult analytical solution than a simple column as ice may fail in a combination of compression, bending and shear depending on the slope of the cone. Michel (1970) uses the results of Korzhavn (1962) to compare the forces resulting from crushing, bending and shearing failure of the ice. For structures with faces at angle θ from the vertical over about 30^{o} it is thought that bending is the predominant mode of failure.

An approximate analysis of ice sheet-cone impacts has been made (Afanasev, et. al., 1971) based on elastic plate theory. This theory requires the calculation of a "characteristic length" as well the length of a circumferential crack thought to develop around the cone. The forces calculated by this method are very sensitive to ice thickness but insensitive to the diameter of the cone at ice level, a development not supported by test data (Neill, 1976). Yield line theory predicts the dependence of forces on cone diameter and gives results which correspond well with ob-

servation (FENCO, Unpublished Report, 1973).

Danys (1971) applies a theory developed for floating ice sheets by Meyerhof (1962) to estimate the force required to fail the ice along the circumferential crack. This approach tends to give much lower forces than Afanasev's method, approximately 1/3 of that for crushing failure on the same cone and 1/6 of that for a vertical cylinder of the same diameter. Yield line calculations, using plastic rather than elastic theory, give an upper bound on the forces. Model testing has indicated that horizontal forces may increase considerably with high ice velocities. Such velocities involve high kinetic energy which requires more work and higher forces. Ice piling on the slope may cause an additional load on the structure and compression of the ice sheet may increase the flexural strength of the ice. These factors are not considered by Meyerhof's theory and would result in higher horizontal forces than predicted.

Problems encountered with conical structures may include the tendency for broken ice pieces to ride up and create piles on and around the structure; damaging the surface, interfering with the superstructure or freezing to the structure and increasing its effective width. If ice rubble freezes to the structure it may stop ice sheets from riding up the cone and forcing compressive failure rather than bending, multiplying the forces on the structure. In some instances special precautions may be required to eliminate this adfreeze.

Dynamic effects of ice mass collision with sloping structures are considered by Kivisild (1976). Two peaks are expected to occur: the first as the mass imparts an impulse to the structure, thereby changing its momentum; and the second as the ice reaches maximum compression or rides up

on the structure, strain rate decreases and failure occurs.

In addition to the horizontal forces, ice impact also produces vertical forces on a cone which tend to increase the sliding resistance of the structure and may increase stability depending on the geometry of the structure. Because of the many advantages it seems likely that platforms used to produce offshore oil supplies in severe ice conditions will incorporate conical shapes.

4. Forces Exerted by Pressure Ridges on Cones

There are few reports in the available literature which analyze forces imposed by collisions with pressure ridges. For some Arctic areas of interest to petroleum developers the largest forces which may be applied to an offshore platform are those due to pressure ridge impact; hence considerable effort will likely be put to the task of this analysis.

Ralston (1977) and Croasdale (1975) consider a pressure ridge to be an infinite elastic beam on an elastic foundation, using the theory of Hetenyi (1946). This theory predicts the vertical forces required to crack the deflected ridge and also to cause secondary hinge cracks in the two resulting ridge pieces. These hinge cracks, required to clear the ridge pieces around the structure, are predicted to require higher vertical forces than that required to develop the original ridge crack. There may also be the possibility of both ridge pieces riding up on the structure simultaneously, causing twice the load.

If the same model is applied to ridges of finite length the vertical force required to cause failure increases for shorter ridges. This theory predicts extreme forces for very short ridges but other failure modes may occur: the ridge pieces may slide over the structure sides without forming a hinge crack or the ridge may not break at all, the surrounding sheet failing instead.

Bercha and Stenning (1979) use the above elastic beam theory for pressure ridges in combination with the previously mentioned plate theory for ice sheets to estimate forces due to ridges embedded in a sheet

of ice. The highest forces were found to occur for relatively short pressure ridges embedded in thin ice sheets.

The above analyses of pressure ridge-structure interaction can only be taken as first order approximations until reinforced by experimental evidence. Model tests of ice ridge impact have shown failure modes which agree well with the above analysis (Lewis and Croasdale, 1978). Full scale tests have not yet been done; this seems to be the next logical step in the further advancement of the analysis.

5. Time Dependent Dynamic Ice Forces

The magnitude of dynamic forces due to ice impact is governed primarily by the ice mass, its loading rate and the controlling failure mechanism. Various authors have reported oscillatory forces on structures. Maattanen (1975) reported violent vibrations of lighthouses during the ice action leading to secondary structural failures, with one structure totally collapsing due to ice pressure. Blenkarn (1970) and Peyton (1968) reported oscillating forces on production platforms in Cook Inlet due to ice moving at both high and very low speeds. Some of the oscillating force may be due to the response of the structure to random ice forces but it is possible that ice crushing at slow velocities may exhibit characteristic vibrations with periods on the order of one second. Such a case could be of great concern as the resonant frequencies of many large structures are similar. The lighthouses mentioned above vibrated at frequencies of .8 - .9 Hz with resulting displacements up to 20 cm when acted upon by thick ice or pressure ridges. These vibrations were near the first natural frequency of the structures. Peyton (1968) notes that very slow velocities produced the largest forces and most severe oscillations of test piles. By using test piles of various resonant frequencies he shows that the oscillating ice failing frequency is independent of the resonant frequency of the structure; hence probably due to ice properties. The oscillations produced by ice failure are dependent only on the properties of the ice sheet itself. Kivisild (1968) examines both slow and rapid force oscillations caused by eccentric loading (oscillating with the period of the free vibrations of the ice sheet as a

membrane) and by the failure mechanism of the ice respectively.

Ice forces on inclined structures are intermittent, both from ice sheets and from masses of floating ice. Tryde (1975) showed that ice sheets failing in flexure on an inclined face are likely to produce an intermittent force pattern. Kivisild (1976) found that a moving ice mass characteristically has two peaks of action. The first short dynamic load changes the momentum of the mass and the second occurs when the total kinetic energy is spent, driving the mass to its highest point on the structure. The relative size of these two force peaks depends on the size and velocity of the ice mass and its strength. Further work is required to develop and test the theories of dynamic action and it is likely that full scale tests will be required to completely assess the time dependent forces which a structure may face in extreme ice conditions.

6. Maximum Ice Forces on a Monocone in the Beaufort Sea

From the literature previously cited it is possible to make very rough first order calculations of the maximum ice forces which a structure may encounter. The rough dimensions of the monocone under consideration are given in Figure 2. The extreme ice conditions expected are approximately as follows:

Ice Sheet :	Thickness	h = 2 m
	Effective Pressure	P_e = 2500 kPa (360 psi)
Pressure Ridge:	Depth	h = 35 m
	Width	w = 120 m
	Flexural Strength	s_f = 700 kPa (100 psi)

A conservative estimate for forces imposed by the ice sheet crushing around the vertical column of the structure can be obtained from the formula recommended by the Canadian Standards Association (1978):

$$F = P_e hd = 75 \text{ MN (8500 tons)}$$

While this is a conservative value for the time-averaged force on a structure with an aspect ratio (d/h) of 7.5, there are likely to be higher local pressures, possibly as high as 8 MPa (Kivisild, 1976, Croasdale, 1975). Pressures as high as 24 MPa may occur if confined compression is the failure mode (FENCO unpublished reports). It is these local pressures which will determine the thickness of the outer shell of the structure.

The previously described method of considering pressure ridges as infinite elastic beams on an elastic foundation leads to the following estimates of vertical forces on the cone (Ralston, 1977):

Initial Crack: $F_v^{\infty} = 4I\sigma_f/y_t\ell$

Hinge Crack: $F_v^{\infty} = 6.2I\sigma_f/y_b\ell$

where I is the cross sectional moment of inertia, y_t and y_b are the distance from the neutral axis to the top and bottom of the ridge respectively, and ℓ is the ridge characteristic length of elastic theory.

$$\ell = \sqrt[4]{\frac{4EI}{k}}$$

where E is the elastic modulus of the ice and k is the foundation modulus.

For the extreme ridge, assuming a triangular shape and an elastic modulus of 5000 MPa, the characteristic length and vertical forces can be calculated as follows:

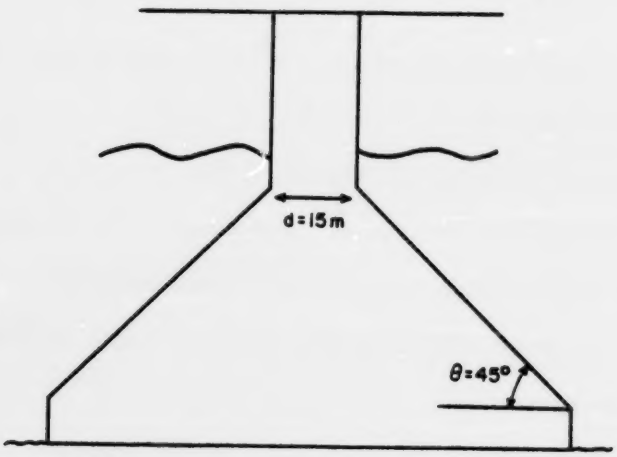

Figure 2 - Approximate Dimensions of Monocone

Calculation of Ridge Loads

$$\ell = \sqrt[4]{\frac{4EI}{k}}$$

$$= \sqrt[4]{\frac{4 \times 5000 \times 1.4 \times 10^5}{1.2}}$$

$$= 220 \text{ m}$$

$E = 5000$ MPa

$I = \frac{wh^3}{36} = \frac{120 \times 35^3}{36} = 1.4 \times 10^5 \text{ m}^4$

$k = \text{density} \times w = 10^4 \text{ N/m}^3 \times 120$

$= 1.2$ MPa

Initial Crack:

$$F_v^{\infty} = \frac{4I\sigma_f}{y_t \ell}$$

$$= \frac{4 \times 1.4 \times 10^5 \times 7 \times 10^5}{10 \times 220}$$

$= 170$ MN (19000 Tons)

Neutral axis is at C of G at depth of about 10 m

$y_t = 10$ m

$y_b = 35$ m

Hinge Crack:

$$F_v^{\infty} = \frac{6.21\sigma_f}{Y_b \ell}$$

$= 260$ MN (29000 Tons)

These extreme vertical loads would be acting on a relatively thin sector of the cone and will put severe constraints on the design. If the collision geometry allows the two ridge pieces to ride up on the structure and form hinge cracks simultaneously the forces on the surface of the cone may be much larger. The vertical forces are also larger if the above theory is applied to ridges of finite lengths.

The increases in vertical force are shown in Figure 3 (Ralston, 1977). This theory obviously does not apply for very short ridges where a different type of failure mechanism must be considered.

The horizontal forces which are required to lift the ridge to the point of failure must be withstood by the structure. These forces are dependent on cone geometry and friction between ice and cone.

$$F_H = F_v (\mu + \tan \theta)$$

For a 45^o cone and an ice sheet not frozen to the cone surface (assuming no friction) the horizontal force would be about equal to the vertical force. For a steeper cone or an ice sheet frozen to the structure the forces could be much higher. Coatings which reduce friction and ice adfreeze as well as gradually sloping sides can reduce the horizontal force considerably.

The maximum horizontal forces may be imposed by short ridges which do not break in flexure. If such ridges hit the structure symmetrically the possibility exists of the ridge grounding on the structure and the ice sheet crushing on the large area of the ridge rather than on the structure. This phenomena has previously been considered for large ice floes or icebergs rather than pressure ridges (FENCO, 1976). A very short ridge on the order of 1/2 the characteristic length is unlikely to fail in flexure on the cone. It is likely that the ridge would collide unsymmetrically and rotate around the cone but the possibility exists of the ridge remaining in a stable position on the cone, with the ice sheet failing around the ridge. The effective pressure over such a large area would be lower, likely below 1500 kPa (220 psi). In such

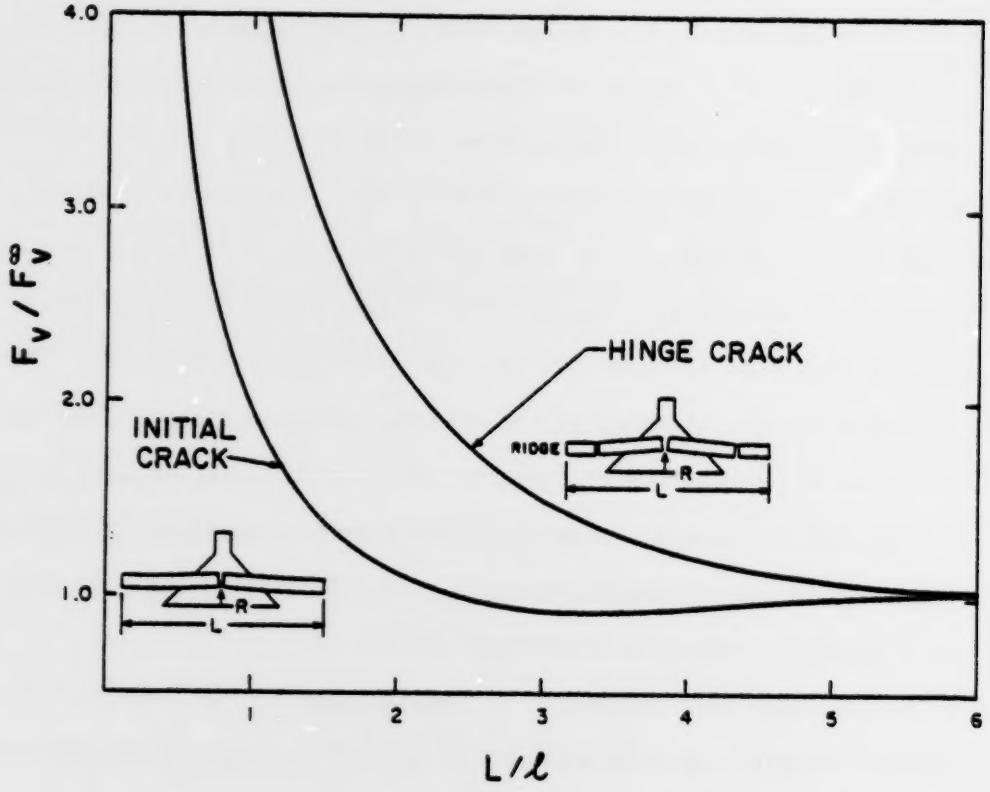

Figure 3 - Elastic interpretation of initial crack and hinge crack. Forces for multi-year ridges failing against conical structures (Ralston, 1977).

a case the horizontal force on the structure, assuming a 2 m thick ice sheet is:

F = 1500 kPa X 110 m X 2 m = 330 MN (38000 Tons)

The two major design problems which this large horizontal force creates are the possible overturning of the structure and the possibility of the ice hitting the superstructure. The above considered ridge could ride high on the cone and if the superstructure is hit it is unlikely to survive. The vertical force of such riding up should help to stabilize the cone but its stability is still critical. These factors must be considered in the design of the structure. The slope of the sides, the height of the apex (above or below waterline) and the strength of the cone all have to be considered in relation to ice forces. The design of any offshore structure in the Arctic is highly dependent on a reliable prediction of these forces.

The above calculations give reasonable approximations to extreme values. However, much work is yet to be done to increase the confidence placed in design criteria for such an Arctic structure. Further experiments, preferably full scale tests, are required to validate this theory, to give reliable values of ice pressures and to further define the failure modes of ice.

Some of the assumptions used here may be changed by test results. The shape of large multi-year ridges is likely not triangular and such ridges may have material properties varying with depth, changing the level of the neutral axis (Weeks, et. al., 1971, Wright, et. al., 1978). The strength values used above may also be changed by future investigations. These questions must be answered before more accurate design criteria can be provided, requiring field tests of ice properties and features.

7. Experimental Methods and Problems

The numerous methods and experiments which are used can be divided into four broad categories as follows:

(1) Testing of material properties.

(2) Small scale model tests (scaling factor 25 - 100).

(3) Large scale model tests (scaling factor 4 - 10).

(4) Full scale structure tests.

The testing of physical properties of sea ice is a complex field. The tests commonly done in the field include unconfined compressive strength (flaking strength), confined compressive strength, flexural strength, coring, salinity, temperature, grain size and crystallography. There is a strong correlation between ice strength and brine volume, dependent on salinity and temperature. Flaking tests are done in pits cut in the ice surface using hydraulic pressure to move plates, usually triangular in shape, against the ice surface. Confined compressive strength is determined using a FENCO borehole jack or similar equipment. This is very useful as strength tests can be made at various depths through the ice (Kivisild, 1975). These and other on site tests can quickly give a good indication of the ice properties. For more detailed laboratory tests an ice core is usually taken and shipped to the lab. The shipping poses problems such as brine drainage or temperature changes. Laboratory tests can accurately determine the material properties of the sample but field tests are highly desirable for comparison. There are not yet standard methods of testing ice

(i.e. a common strain rate) as there are for many other structural materials.

Small scale model tests have been reported by numerous authors. These tests are relatively inexpensive and can be done with quite simple equipment. The most common test is done by "growing" the required ice in a tank and moving an instrumented structure through the ice. For accurate scaling effects the model ice must be considerably different from real sea ice, with strength and modulus scaled the linear scaling factor and density and friction coefficient unchanged. These conditions can be approximated by ice of very high salinity or by synthetic ices. Edwards, et. al. (1971) tell of the complexities of building and using such a model testing facility.

Pearce and Strickland (1979) report the making and testing of a synthetic ice using various conical structures. Murat and Tinawi (1977) tested ice in flexure and Sackinger (1977) tested the strength of the adfreeze bond using a model tank. Schwarz, et. al. (1974) tested strain rate, column diameter and ice thickness effects on compressive failure forces using small scale models. Lewis and Croasdale (1978) describe the testing of ridge loads on structures using small scale models and saline ice.

The two major problems encountered in model tests are the preparation of a synthetic ice with accurate and reliable scale effects of physical properties, and extrapolation to full scale values. Maser (1971) discusses the use of small scale results. Determining full

scale forces on a structure from model tests is very difficult due to the wide range of sea ice conditions, the uncertainty of scaling errors and other differences between the laboratory and field conditions.

Large scale model tests have become popular as a compromise having less problems with scaling than small scale tests but still lower costs than full scale tests. Large scale test may be done in test basins such as described by Robbins, et. al. (1975) where ice thickness, strength and strain rate can be controlled, in lakes where costs are quite low or on the actual site where costs are very high but the actual ice forces are experienced. Large scale tests such as the nutcracker tests reported by Croasdale (1974) give a good indication of the average forces which may be experienced by a structure. At the present time it seems likely that further on site large scale tests will be required to more accurately determine the forces which a full scale structure must withstand in severe ice conditions.

8. Full Scale Tests of Instrumented Structures

There have been numerous tests of full scale instrumented structures in Sub-Arctic areas. These include test structures and production platforms in Cook Inlet (Blenkarn, 1970), Canadian offshore lighthouses (Danys and Bercha, 1975), European lighthouses (Engelbrektson, 1977, Maattanen, 1975) and a wharf in the Canadian Arctic (Frederking and Sinha, 1977). Unfortunately none of these structures have been in ice conditions as severe as those in the Beaufort Sea or off the Labrador Coast. The results are nevertheless very useful and should be used in determining the test structure to be used in the Canadian Arctic. The performance of the platforms in Cook Inlet has shown that large structures can withstand quite severe ice conditions and other environmental forces. The pressures on the legs of these platforms due to crushing ice have been measured on the order of 1 MPa (150 psi) or less. Oscillating forces measured on these platforms may be a significant factor in the design of a test structure.

The forces measured on lighthouses have contributed much to the knowledge of ice mechanics and the design of pressure measuring panels for these structures may apply directly to an Arctic test structure. A drawback to most ice force measurements on lighthouses is the difficulty of obtaining ice strength and thickness estimates from the unmanned facilities. The force-time data acquired is useful for statistical analysis but very difficult to apply to other areas with different ice conditions.

Measurements of ice forces on wharves and observations of ice

action on the artificial islands in the Beaufort Sea are hampered by the fact that the ice conditions in shallow or inshore waters are significantly different from those further offshore. In fact no direct measurements of ice forces have been made for the critical ice features which govern the structure of Arctic platforms. No fixed structure has withstood the impact of a thick multi-year pressure ridge, a massive multi-year ice floe, an ice island or iceberg.

The next task then seems to be to design a structure which will withstand and measure the forces of the predicted ice features. This test production structure will undoubtedly be very expensive to build and install but is required to prove the capabilities of an Arctic platform and to validate the theory used in the design. While such a structure is unlikely to experience the maximum forces it should establish the theory well enough to give accurate predictions of these extremes.

9. Summary and Conclusions

The search for offshore oil and gas supplies has added impetus to the development of both theoretical and practical knowledge of ice forces on structures. The general form of ice failure mechanisms are known well enough to predict forces on simple structures with reasonable accuracy and enough to predict forces on simple structures with reasonable accuracy and reliability. The theory and experiments evaluating forces on circular columns from ice failing in compression are well documented. The main variables yet to be resolved are those which are highly site specific. It is unlikely these questions will be answered away from the area of concern. The production platforms in Cook Inlet provide an example of structures standing up well to forces of ice failing in compression.

The loads imposed by ice sheets failing in flexure on conical structures can be predicted using theories giving both upper and lower bounds. A conservative value for these forces can be given for various cone shapes and ice sheet properties. Model tests and the experience of offshore lighthouses can be used to back up such predictions. While no full scale tests of forces imposed by large pressure ridges are available the theoretical predictions are in agreement with model observations and are considered reliable. Maximum forces for cone structures in the Beaufort Sea are calculated to be 330 MN or 38,000 tons. This force considers the interaction of the cone, large ice masses and the ice sheet.

The dynamic forces of ice impacts are not yet well explained in

the published literature but a rough estimate of force-time relations can be given from theory, tests and the experience of other structures. Much of the required information seems to be site specific. Development work in this area is presently being conducted by FENCO.

Both large and small scale tests have contributed greatly to knowledge of ice forces. Theory together with experimentally determined ice properties give results which can be compared with such model tests. Full scale tests are the final step in giving a reliable determination of the forces imposed by specific ice features on a specific structure in a specific area. The combination of these various methods is capable of producing the design criteria for a permanent offshore structure.

10. Recommendations

Before full scale production of hydrocarbons in the Beaufort Sea can commence on a commercial basis a test production platform will likely be required to prove the reliability of the structure.

The normal progression of events leading to a large structure is: (1) small scale model tests, (2) large scale tests, (3) full scale test structure, and (4) full scale operations. The first two steps have already been taken for the Beaufort Sea production platform. Due to the extreme costs it may be possible to combine a full scale test structure with an initial operating platform. If further large scale tests are required to test the specific design concepts of the structure, they can readily be accomplished at present test facilities and sophisticated facilities presently being developed. Full scale tests in less severe ice conditions are expensive and of limited usefulness and are unlikely to provide the answers to all the problems faced in the Beaufort Sea.

Further development of the theory of dynamic forces and study of existing structures is likely to be of great use in designing durable production platforms. FENCO is at present developing such techniques for Japanese interests. For site specific information some further field tests may be required to further define the ice features and properties. The most promising path, however, seems to be the development of a full scale test production platform to prove the viability of a structure in the Canadian Arctic.

REFERENCES

Afanasev, V.P., et. al., (1971), "Ice Pressure on Individual Marine Structures", Leningrad, Translated by Israel Prog. for Sci. Trans., Jerusalem.

Bercha, F.G. and Stenning, D.G., (1979), "Arctic Offshore Deepwater Ice-Structure Interactions", Offshore Technology Conference Paper No. 3632, Houston.

Blenkarn, K.A., (1970), "Measurement and Analysis of Ice Forces in Cook Inlet Structures", Offshore Technology Conference Paper No. OTC 1261, Houston.

Canadian Standards Association, (1978), "Design of Highway Bridges", CAN3-S6-M78.

Croasdale, K.R., (1974), "The Crushing Strength of Arctic Ice", A.I.N.A. Symposium on Beaufort Sea Coastal and Shelf Resources, Montreal.

Danys, J.V., (1975), "Effect of Ice and Wave Forces on the Design of Canadian Offshore Lighthouses", Can. J. Civ. Eng., Vol. 2, No. 2.

Danys, J.V., (1971), "Effect of Cone Shaped Structures on Impact Forces of Ice Floes", POAC Conference, Norway.

Danys, J.V., and Bercha, F.G., (1975), "Determination of Ice Forces on a Conical Offshore Structure", POAC Conference, Alaska.

Edwards, R.Y., et. al., (1971), "An Arctic Ice Model Basin-Design, Construction and Operating Experience", POAC Coference, Norway.

Engelbrektson, A., (1971), "Dynamic Loads on a Lighthouse", POAC Conference, Newfoundland.

FENCO CONSULTANTS LTD., (1976), "Labrador Offshore Structure on a Submerged Mound", Submitted to Eastcan Exploration Ltd.

Frederking, R., and Gold, L.W., (1971), "Ice Forces on an Isolated Circular Pile", POAC Conference, Norway.

Hetenyi, M., (1946), "Beams on Elastic Foundation", University of Michigan, Ann Arbor.

Iyer, S.H., (1978), "Existing Ice Codes and Suggested Criteria", I.A.H.R. Symposium on Ice Problems, Sweden.

Kivisild, H.R., (1976), "Offshore Structures in Arctic Ice", Conference on Behaviour of Offshore Structures, Norway.

Kivisild, H.R., (1971), "Year-Round Oil Terminal in Ice Covered Waters", POAC Conference, Norway.

Kivisild, H.R., (1968), "Ice Impact on Marine Structures", 19th Technical Meeting of the Petroleum Society of C.I.M., Calgary.

Korzhavn, K.N., (1971), "Action of Ice on Engineering Structures", U.S.S.R. Acad. Sci.-Siberian Branch, Translated by CRREL.

Lewis, J.W., and Croasdale, K.R., (1978), "Modelling the Interaction Between Pressure Ridges and Conical Shaped Structures", I.A.H.R. Symposium on Ice Problems.

Maatanen, M., (1971), "Experiences of Ice Forces Against a Steel Lighthouse", POAC Conference, Norway.

Maser, K.R., (1971), "Determination of the Strength of an Ice Sheet from Small Sample Data", Offshore Technology Conference, Houston, Arpil, Paper OTC 1424.

Meyerhof, G.G., (1962), "Bearing Capacity of Floating Ice Sheets", Trans, A.S.C.E., Volume 127.

Michel, B., (1970), "Ice Pressure on Engineering Structures", CRREL Monogr. III-B1b.

Murat, J.R., and Tinawi, R., (1977), "Sea Ice-Testing in Flexure", POAC Conference, Newfoundland.

Neill, C.R., (1976), "Dynamic Ice Forces on Piers and Piles", Can. J. Div. Eng., Vol. 3.

Pearce, J.C., and Strickland, G.E., (1979), "Ice Forces on Conical Structures", Offshore Technology Conference Paper No. OTC 3635, Houston.

Peyton, H.R., (1968), "Ice and Marine Structures", Ocean Industry, March, September and December.

Ralston, T.D., (1977), "Ice Force Design Considerations for Conical Offshore Structures", POAC Conference, Newfoundland.

Robbins, R.J., et. al., (1975), "Techniques for the Study of Ice-Structure Interaction", POAC Conference, Alaska.

Sackinger, W.M., et. al., (1977), "Shear Strength of the Adfreeze Bond of Sea Ice to Structures", POAC Conference, Newfoundland.

Schwarz, J., et. al., (1973), "Model Technique for the Investigation of Ice Forces on Structures", POAC, Iceland.

Tryde, P., (1975), "Ice Forces Acting on Inclined Wedges", POAC Conference, Alaska.

Watts, F.J., and Podolny, W., (1976), "Ice Loads on Bridge Piers", U.S. Federal Highway Administration, Structural Engineering Series No. 1.

Weeks, W.F., et. al., (1971), "Pressure Ridge Characteristics in the Arctic Coastal Environment", POAC Conference, Norway.

Wright, B.D., et. al., (1978), "Sea Ice Pressure Ridges in the Beaufort Sea", I.A.H.R. Symposium on Ice Problems, Sweden.

INSTRUMENTATION SYSTEMS FOR A COLD OCEAN TEST STRUCTURE

by

J.M. Jones
Centre for Cold Ocean Resources Engineering
Memorial University of Newfoundland

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Instrumentation Systems for a Cold Ocean Test Structure

1. Introduction

The fundamental function of an instrumentation system for a Cold Ocean Test Structure will be to identify and measure the magnitudes of the environmental forces acting on the structure, to evaluate its performance in respect to assumptions made in its design and to develop ice monitoring and management procedures.

The geographic area of immediate interest is the Beaufort Sea, where there is a requirement for production drilling platforms capable of year-round operation. Water depth in the areas of interest is from 20 to 80 m, too deep for an artificial island and in the region of the mobile polar pack. The greatest environmental hazard to a structure in this area will be the consolidated multi-year ice ridge. These ridges may be in excess of 40 m thick and frozen into and drifting with the polar pack at speeds of up to 3 m/s during spring breakup (Hnatiuk et al, 1978; Wright et al, 1979).

Two types of structures have been proposed by drilling operators to resist the force exerted by these multi-year ridges: the conical monopod and the large caisson type, both of which are intended for year-round production drilling operations.

The conical monopod is to be designed so that the ridge fails in bending as it rides up the conical base of the structure. The very much larger caisson structure resists the ridge force by causing the ice sheet surrounding the ridge to fail either in compression or flexure.

Two types of conical monopod structures have been designed for the Beaufort Sea and have been described in the literature by Stenning

and Schumann (1979) and by Jazrawi and Khanna (1977).

In substantiation of the design of these structures, a number of theories of ridge-structure interaction have been developed by Bercha and Stenning (1979) and by Ralston (1977). In developing these theories various assumptions were made regarding the elastic-plastic behaviour of the ice sheet, the ridge shape and the interaction modes of ridge and structure. Therefore, an important part of the instrumentation study should be directed at the verification of these theories. An equally important part of the study will be the monitoring and prediction of ice movement and the characterization of ice and ridge properties in regions of likely impact. This will involve rapid, in-situ ice testing and the planning of emergency countermeasure activity, such as blasting, if extreme conditions were to occur.

In view of the enormous expense involved in the construction of a full scale structure, it has been proposed that geographical features, such as rocks or small islands, might be used as an alternative to perfect in-situ measurement techniques, study ice movements, validate ridge-structure interaction models, and estimate the ice forces acting on the island. Some work along these lines has already been done by Sackinger and Nelson (1979) using in-situ ice stress transducers placed in the vicinity of grounded floes and pressure ridges. Impulsive compressive stresses of 250 psi. (1.72 MPa) were noted to correspond with ice movements and wind induced tensile stresses of 100 psi. (0.69 MPa) were found to open up cracks in the ice. Further observations concluded that variations in ice stress were confined to 100 m or so around the ground obstacle.

2. Ice Force and Structural Stability

The ice ridge exerts, by far, the greatest environmental force

that will act on a structure. If it can resist the ridge force, then it will almost certainly withstand all other environmental forces.

The instrumentation system developed for the structure should be capable of measuring four characteristic ice forces:

(a) Total resultant ice force and overturning moment

This is the resultant force and moment of all ice forces acting on the structure and is required in order to evaluate the location of the resultant ridge force on the surface of the structure, the force tending to slide the structure sideways along the seabed and the overturning moment.

(b) Large surface force

This is the force exerted by the ice in contact with large sections of the cone. In order for the cone to develop sufficient force to crack a ridge, some localized crushing will need to take place. The ridge force would then be the product of the average strength of the ice and contact area between ridge and cone. This force would be required in order to ensure the local structural integrity of the internal cone support structure.

(c) Localized or puncture forces

This is the maximum force that can be exerted by ice under confined compression and is required in order to ensure that the plating does not fail between frames and that internal members do not push through the outer skin of the structure.

(d) Ridge clearing force

Calculation of ridge clearing forces have been based on assumptions

made regarding shape of the ridge and the motion of the broken parts as they pass on either side of the cone. Bercha and Stenning (1979) assume that parts of the broken ridge will ride up and slide over the surface of the cone while the other end, adjacent to the hinge crack, may be submerged.

In order to validate such assumptions, the instrumentation system should be capable of measuring localized ice forces, in addition to the single resultant force, at a number of sectors around the cone. It should, however, be noted that high frictional forces acting along the surface of the cone complicate the design of force panels mounted on the surface of the cone.

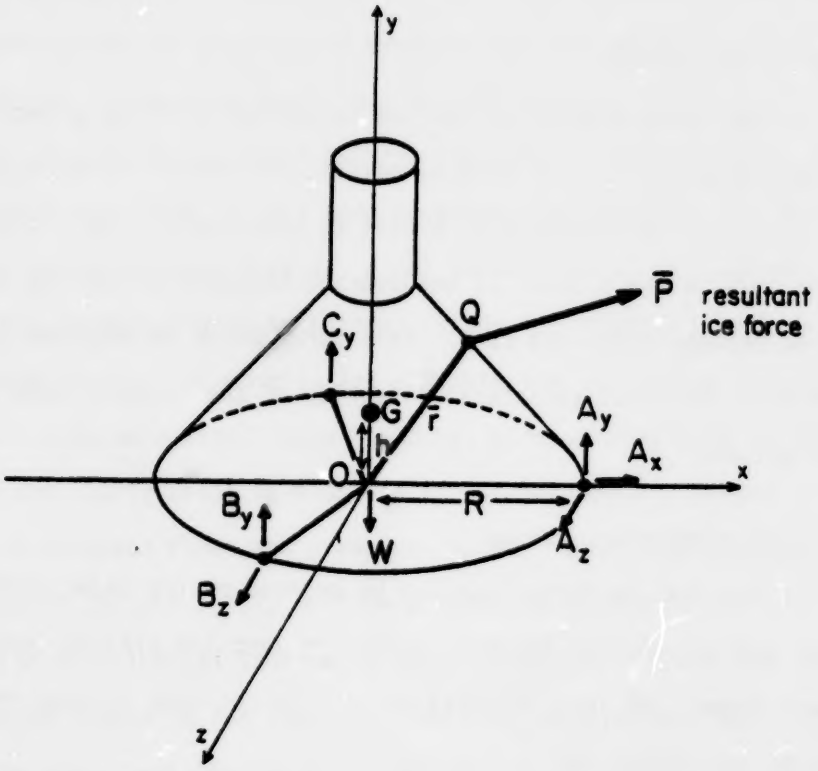

Figure 1 Conical structure on 3-point support with six degrees of freedom

3. Idealized Method for the Measurement of Total Ice Force and Overturning Moment

The following section considers ridge/cone interaction in simplified terms in order to clarify the problem and to investigate the value of isolated measurements in relation to an assessment of overall structural stability.

Consider an ice ridge moving against a conical structure, such as that illustrated in Figure 1, supported at three points A., B. and C. The ridge will exert a varying distributed force on the surface of the structure that is in contact with the ice. As far as the structure is concerned, this distributed ice force is equivalent to a single resultant force acting at a point on the surface which may be termed the center of ice pressure Q.

As the ice may be sliding up or around the surface of the cone, it should not be assumed that this resultant force is normal to the surface of the cone. A friction or adfriction force may cause the resultant force vector to be inclined to the surface and it is therefore represented by the general vector notation $\overline{P}$. Three quantities are therefore necessary to define the magnitude and direction of the resultant ice force $\overline{P}$.

As the ridge moves up or around the cone, the centre of ice pressure will move. The position of the centre of ice pressure Q may be defined by a position vector $\overline{r}$ which is referred to the origin of an orthogonal coordinate system located at the centre point of the base plane.

Three quantities are necessary to define the position vector $\overline{r}$, which brings to six the number of quantities necessary to define the magnitude, direction and point of application of the resultant ridge force acting against the structure.

The resultant of the x and z components of $\overline{P}$ is the force trying to slide the structure sideways across the seabed and the vector product $\overline{r} \times \overline{P}$ is the moment of the resultant ice force about the origin. Note that the direction of vector $\overline{P}$ can affect the moment about the origin significantly.

Six scalar equations are required to define the condition of static equilibrium of a rigid body in the generalized three dimensional case. We are therefore in a position of being able to define $\overline{P}$ and $\overline{r}$ provided we make six <u>independent</u> measurements of the reactions at the supports of the structure.

The equations of static equilibrium are:

$$\Sigma F_x = 0 \qquad \Sigma F_y = 0 \qquad \Sigma F_z = 0$$

$$\Sigma M_x = 0 \qquad \Sigma M_y = 0 \qquad \Sigma M_z = 0$$

Therefore, in order to be able to achieve the ideal situation of being able to measure $\overline{P}$ and $\overline{r}$ we must ensure that the design of the supports of the structure are such as to permit 6 degrees of freedom, each of which is provided with an appropriate load measuring device.

Additional constraints result in a statically indeterminate support. It should be noted that additional constraints might be provided by friction within the load measuring system and efforts should be made to ensure that this minimized and accounted for (Browne, 1975).

The support system illustrated in Figure 1 provides the six constraints necessary to hold the structure in terms of measured reactions A_x, A_y, A_z, B_y, B_z and C_y.

Independent load measuring systems would be necessary to measure each of these six components designed so that there is no interaction between the measurements.

An analysis of static equilibrium will give the following equations:

$$\Sigma F_x = P_x + A_x = 0$$

$$\Sigma F_y = P_y + A_y + B_y + C_y - W = 0$$

$$\Sigma F_z = P_z + A_z + B_z = 0$$

From which P_x, P_y and P_z may be determined directly from the measurements of A_x, A_y, A_z, B_y, B_z, and C_y, W is the weight of the structure acting vertically downwards, from G, through the origin.

Summing the components of the moments about each of the 3 axes leads to

$$\Sigma M_{ox} = -(A_z + B_z)r_y + (A_y + B_y + C_y - W)r_z + .87R(C_y - B_y) = 0$$

$$\Sigma M_{oy} = -A_x r_z + (A_z + B_z)r_x + .5R(B_z - 2A_z) = 0$$

$$\Sigma M_{oz} = (W - A_y - B_y - C_y)r_x + A_x r_y + .5R(2A_y - B_y - C_y) = 0$$

which may be solved simultaneously to give r_x, r_y and r_z.

If a real, bottom founded, gravity structure is considered, it is not possible to make independent measurements of the foundation reaction with consequent limitations to the isolation of individual orthogonal structural forces. For instance, the horizontal structural force calculated from in-situ stress measurement made in the ice sheet surrounding the structure can only lead to the total horizontal force acting on the structure or in other words, only two of the six quantities necessary to define the static stability of the structure the overturning moment could be as estimated by assuming the resultant ice force acts at the midpoint of the ice thickness. However, if a ridge acts against the cone, the location of the resultant force cannot be readily defined.

Overturning moments acting on the structure could be estimated by measuring strains in the structure itself with an embedded strainmeter in a manner similar to that carried out by Maattanen (1977) or by geophysical measurements of pore-pressure and total earth pressure under the structure as carried out by Bourgeois and Leroy (1976). Both of these methods utilize the structure itself as the transducing element and therefore require extensive structural analysis or sediment "calibration" in order to establish the sensitivity of the structure to external forces. Such measurements would give very useful information on structure performance but would be a very indirect method of evaluating ice forces.

4. Geophysical Instrumentation

In the design of the full scale, bottom founded or piled structure it will not be possible to support the whole structure on an instrumented three point support. The six equations of static equilibrium must, however, be satisfied, so the monitoring of overall structural stability becomes the task of the foundation instrumentation and instruments attached to the structure itself such as strainmeters, tiltmeters, seismometers, vibration instruments etc.

The seabed in the Beaufort Sea presents a difficult foundation problem due to unconsolidated sediments, and embedded permafrost layers. Prior to recommending any geophysical instrumentation, it will be necessary to conduct extensive geotechnical investigations at the proposed site and develop a numerical model of the structure seabed interaction in a manner similar to that described by Hobbs et al (1978) for structures in the North Sea. A numerical model of the system will permit the estimation of the magnitudes and location of settlement strains under the

structure, pore-pressure distribution and total earth pressure. Given this information, appropriate instrumentation can be selected and deployment methods developed for the particular site. Bourgeois and Leroy (1976) outline some of the methods used for structures in the North Sea and other standard methods of foundation instrumentation are described by Hanna (1973).

5. In-Situ Ice Force Measurement

A number of techniques have been developed for estimating the compressive force within the ice sheet. Allan and Winsor (1977) describe the use of a wire strainmeter which has proved effective for the measurement of principal surface strains in an ice sheet but is sensitive to flexural as well as direct strain. Various types of embedment strain gauge are commercially available which can be frozen-in at different depths in the ice. e.g. if the gauge is set at the neutral axis then only strains due to the direct force will be measured.

The greatest problem with the use of strainmeters is that a knowledge of the modulus of ice is required in order to calculate the stress or force distribution. The magnitude of the modulus can vary significantly with ice type, temperature, salinity, crystal orientation and, in particular, porosity (Michel, 1978).

In an attempt to overcome problems due to variations in the modulus of ice, various types of stressmeters have been developed by Metge et al (1975), Nelson (1975), Nelson et al (1976) and Templeton (1979). These stressmeters are proposed solutions to the problem, defined by Coutinho (1949), relating to the measurement of stresses by embedded transducers having a different effective modulus to that of the surrounding material.

The presence of the transducer creates a stress concentration of indeterminate magnitude; the transducer then measures the magnified stress. Nelson (1975) discusses the magnitude of the stress concentration and how its dependence on the modulus of ice can be minimized by the appropriate selection of transducer size and stiffness.

Stressmeters are installed by freezing into slots or cavities cut in the ice. Some time is then required for the ice to "creep" around the inclusion and apply the full ice force to the measuring element. Therefore they are not all that well suited to rapid emplacement in moving ice conditions.

Such devices can give a useful measure of the force field pattern in an ice sheet. However, where the analysis of the ridges force against a structure is concerned, they can only give a very indirect measurement of the force in the ice sheet pushing the ridge, which can then be reduced to the horizontal component of the ridge force or only two of the six measurements necessary to define structural reaction.

6. Location of Stress and Strainmeters on a Uniform Ice Sheet

The objective of measuring strains in the ice sheet surrounding a structure is to deduce the force acting on that structure. Provided the ice sheet thickness is constant, there are essentially two areas of interest in the stress field - the near field and far field.

The structure acts as a loading point on the ice sheet with the result that there is considerable variation in stress immediately ahead of the structure. Measurements made in this area are within the "stress concentration" area of the loading point, where the ice is subject to large strain gradients and strain rate effects which can induce plane

strain fracture or ductile failure, depending on the prevailing conditions. In general, measurements in this region would present difficulties both from the point of view of interpretation and the possible destruction of the measurement system. A theoretical assessment of ice failure in this region is given by Frederking & Gold (1971).

At some distance from the structure, approximately three or four structure diameters, the local stress field due to the structure should tend to the idealized stress field which could be represented by a point load acting on the boundary of an infinitely large plate (St. Venant's principle). The analysis of far field strain would be much simpler yet would contain the same information as the near field in respect to measuring the horizontal force acting on the structure, provided creep strains are small compared with the strain produced by the reaction of the structure. The far field region would not be subject to the same high strains, strain rates or strain gradients as the near fields, therefore it might be possible to analyse the field using standard elasticity theory as outlined by Timoshenko and Goodier (1951). Using sensitive strainmeters it should be possible to measure very small strains and also have a much longer time period over which the strain meter is usable.

It is therefore suggested that further investigation be carried out with the objective of defining the approximate boundaries of near field and far field conditions in an uniform ice sheet, and to determine the extent to which simple linear elastic theory can be applied to far field strain measurements.

The practical application of stress and strainmeters in connection with ridge interaction requires that procedures are developed for spec-

ifying the location and orientation of the gauges relative to the ridge and the probable point of impact. Analytically this presents a difficult problem because ridges do not conform to any particular shape or strength. It is theoretically possible to define the stress field associated with a point load in a uniform ice sheet, but the analysis becomes extremely complex due to stress concentration caused by the presence of a stiff ridge within the sheet. It is therefore suggested that photoelastic modelling techniques be used to develop experimental guidelines for locating strain gauges within the area of influence of the "stress concentration" so that representative measurements are taken. Measurements made outside the area of influence of the ridge tend to the average ice sheet stress. This has been verified by Sackinger and Nelson (1979) who noted that ice stresses in the vicinity of grounded obstacles are localized to a range of the order of 100 m.

7. Ridge - Structure Interaction

The analyses developed so far for ridge/structure interaction assume the ridge to act as a beam supported on an elastic foundation. The analysis of the vertical force required to crack such a beam has been carried out by Ralston (1977) and Bercha and Stenning (1979).

An extension of this model would be to include the horizontal component of the resultant force and the torsional stress acting on the beam in addition to the flexural stress.

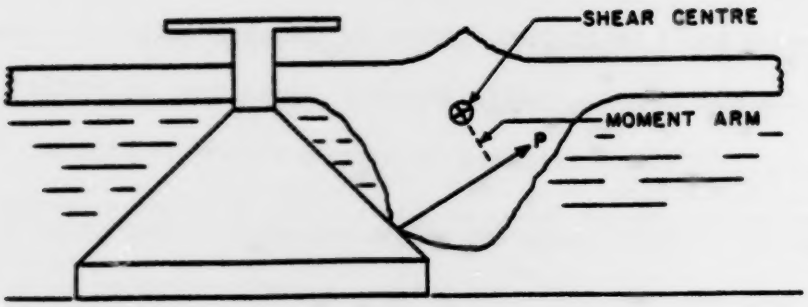

Figure 2 Ridge/Structure Interaction

In order to carry out such an analysis, one would need the approximate location of the "effective" shear centre of the ridge which in turn would require a knowledge of the strength and modulus distribution of the whole cross section of the ridge. The torsional stress would combine with the flexural stress in a manner which might alter the failure model proposed for the ridge.

It is possible to propose other models of ridge/structure interaction for which the ridge is not broken in flexure . Ridges are not always straight and it is conceivable that a suitably shaped ridge, multiple ridge or thick floe would clear the structure with a rotating, sliding motion around the core that might require the failure of an extended surface of the restraining ice sheet, the net result being that effective projected area of the structure and interacting ridge is much greater than the geometrical size of the structure itself.

In assessing the safety of conical offshore structures there are a multitude of possible interaction modes, some of which have been examined analytically and others which have not. It is therefore somewhat premature to state that a structure capable of breaking a straight ridge in flexure

is safe for all other conditions. The possibility of other interaction modes occurring could be studied further by the observation of ridge movements around small islands, rocks, grounded ice-islands or similar features.

8. Force Panel Measurements

The design of force measuring panels represent a compromise between incomplete data and load cell complexity. A similar problem exists in the field of soil mechanics and no simple solution has yet been developed.

In considering the general case, an inset force panel at the structure/ice interface is subject to a resultant force which can act anywhere on the surface of the panel. This force can be resolved into a normal component and a transverse or shear component that can act at any angle in the plane of the panel. In order to define this resultant force, three quantities are needed and a further two quantities are required to define its point of application in the plane of the panel. Therefore a minimum of five dependent quantities are theoretically necessary to define the force acting on the panel.

A simplified hinged panel utilizing one load cell, as used by Neill (1970), can measure the normal component of the resultant force, only if an assumption is made as to the point at which the resultant force acts. The transverse and rotational reactions, which are provided by the hinge, are not measured.

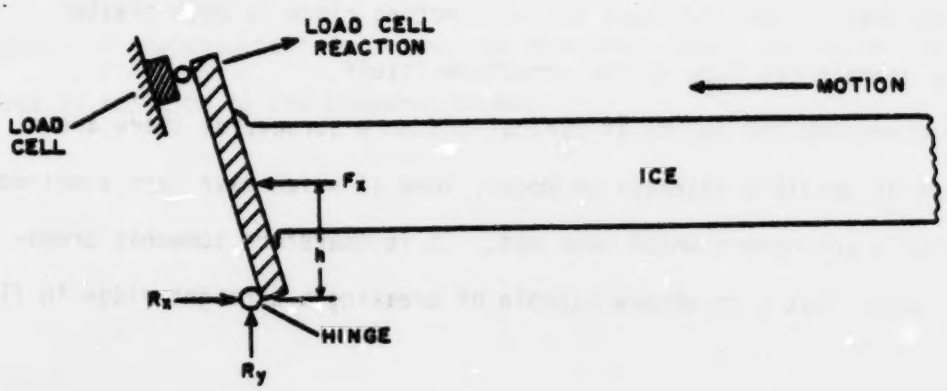

Figure 3 Ice force measured with single load cell

Similarly with panels supported on 4 load cells, Danys and Bercha (1975), the load cell outputs are not independent measurements. The sum of the outputs gives a measure of the total normal force, provided the transverse shear component along the panel did not introduce a cross-sensitivity error in the load cells.

The design of an ideal force measuring panel presents practical problems of providing the required number of degrees of freedom and the complexity of mounting six load cells such that they do not interact with one another. However, such a design is possible and laboratory versions of such instruments are described by Hanna (1973) and Prange (1972).

Some practical problems associated with the operation of force panels are discussed by Frederking and Sinha (1977). Problems that have been encountered include the bridging of the ice across the sensitive area of the load cell and the rocking or angular change of the ice sheet in contact with the panel due to tidal and ice movements. Shear forces at the ice/panel interface, which are generated by a friction or adfriction mechanism, can be of considerable magnitude. Sackinger and Sackinger (1977) measured adfreeze bond strengths of over 0.5 MPa, the actual magnitudes varying with salinity and temperature. The force panels should be designed to measure, or at least withstand, this force without producing an error in the normal force measurement. If shear forces of such magnitudes are unacceptable, then some consideration may be given to heating the panels.

General conclusions regarding panel design would indicate that force panels would need to be large with boundaries removed from fixed supports

in order to prevent bridging of the ice over the panel. If adfriction takes place, significant shear forces will be induced which will vary with ice movement and tidal fluctuation. These are naturally occurring forces and should be measured if at all possible. It is therefore proposed that further studies be carried out into the design of large force panels which are suitable for use on a full scale structure and which have the capability of measuring the shear force component in the plane of the panel.

9. Ice Ridge Measurements

In order to conduct meaningful experimental work as a check on analytical methods of predicting the force that can be exerted by an embedded ice ridge, it is essential to measure the strength characteristics of the ice ridge, the surrounding ice sheet, the dimensions of the ridge and dynamic ice conditions before, during and after the ridge/structure interaction.

The physical dimensions of a ridge such as length and axial shape can be measured on the ice surface using conventional survey techniques. The underwater profile can be measured using a side-scan sonar or upward looking sonar. The biggest problem is to obtain a two dimensional strength profile of the ridge at the point where the ridge is going to strike the structure and at the location of the hinge cracks.

The quantities of interest, from an analysis point of view, are the tensile and crushing strength of the ice. It is also highly desirable to have a measure of the degree of consolidation of the ridge, its salinity and temperature in order to rank the ridge in terms of its severity in relation to the worst ridge that is likely to be encountered by the structure over an extended operating period.

The need for a rapid, in-situ, method of obtaining a ridge strength profile suggests that borehole test techniques should be examined in relation to ice testing. Many of the available techniques were developed in relation to petroleum production and rock mechanics studies and have not, as yet, been applied to any great extent to the study of ice.

The only borehole tests that are a direct measure of strength are uniaxial jack tests, using Goodman type jacks, dilatometer tests, using a variation of the Ménard pressure-meter used in soil mechanics studies and the hydraulic fracturing techniques which is used widely for oil well stimulation.

Uniaxial jack tests are carried out using a special hydraulic jack which fits inside the borehole and which has two rams set diametrically across the hole. The test is carried out by measuring the force required to drive the rams into the sides of the borehole. Due to the complex stress distribution around the rams of the jack and the anisotropic nature of the ice such equipment requires calibration in the laboratory. It should then be possible to conduct meaningful field tests using this technique.

The dilatometer or Ménard pressure-meter has been used by Ladanyi (1978) to measure creep in ice, but has not as yet been used for measuring failure stress in ridges. The theory behind the use of these instruments is well developed and would appear to be ideally suited for ridge strength profile measurements. The use of dilatometers and inflatable flatjacks for deformability testing of rock is described by Rocha (1969) and it would appear that a number of these techniques could be adapted for ice testing.

Hydraulic fracturing is a technique used to increase the production rate of an oil well by using hydraulic pressure to fracture and "open-up" a producing formation. The formation to be fractured is isolated by means of inflatable straddle packers and a fracturing fluid pumped into the section between the packers. The hydrostatic pressure required to fracture the formation gives a measure of the tensile strength of the formation. If the pump is stopped after the breakdown and the well shut-in, the instantaneous pressure is a measure of the pressure required to keep the formation fractures open and therefore is a measure of the in-situ stress and the pressure force required to propagate a crack through the formation. The leak-off rate after shut-in gives a measure of the formation porosity. Excessive porosity in the formation preventing the build up of pressure can be counteracted by using a more viscous fluid.

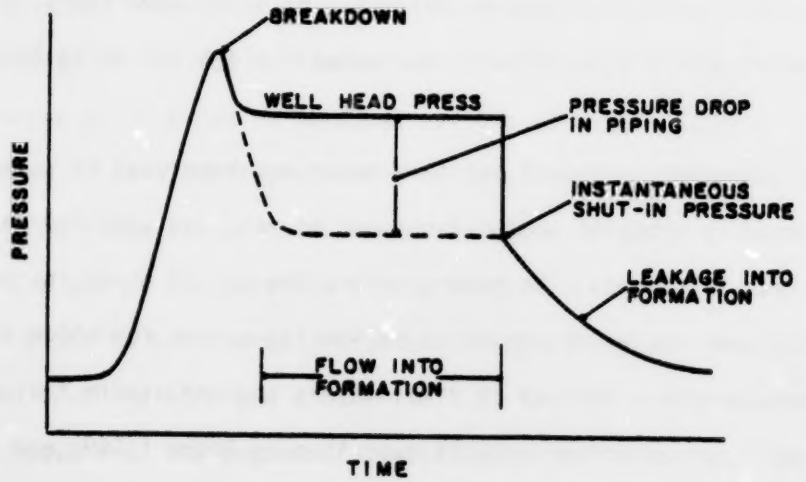

Figure 4 Typical pressure time curve for formation fracture

The information provided by a hydraulic fracture test, namely tensile strength and porosity, is precisely the information that is required for a ridge that is failed in bending. Therefore, it is strongly recommended that a study be undertaken to establish whether this technique can be applied to ice. It is doubtful if conventional oil well tools would be suitable due to their size, the extreme environmental conditions, and the need to operate in an open or uncased hole. However, suitable portable equipment such as packers, fluids and power supplies could be designed specifically for ice testing.

The theory of flow stimulation by fracturing is well established and is described by Allen and Roberts (1978) and by Howard and Fast (1970). The latter reviews the various theories of failure that have been proposed and it would be particularly instructive to study these experimentally in relation to the failure of ice.

A study of hydraulic fracturing applied to the in-situ measurement of stresses in rock is described by Haimson (1973) who discusses the orientation of fracture planes, in relation to the relative magnitudes of the principal stresses, and how their direction may be measured with impression packers and an acoustic televiewer. The possibility of producing extended fractures in thick ice should be investigated further because it opens up a number of possibilities regarding the weakening of ice ridges and the break-up of large masses of ice.

In addition to the measurement of ridge strength it is also going to be necessary to monitor the dynamics of ridge movement around the structure in response to environmental conditions such as wind speed, current and ice drift. This can be accomplished, at some distance from

the structure, by mounting radar transponders on the ice and photographing the radar screen with a time lapse camera. The motion of the ridge as it comes up against the structure could be monitored using time-lapse stereo-photography. This would provide a quantitative description of ice ridge movement which can be compared with the mathematical ridge failure models proposed by Bercha and Stenning (1979) and by Ralston (1977).

10. Instrumentation System

In reviewing the number of measurements that might be required for the instrumentation of a test structure, it is evident that masses of raw data will be produced which will tend to saturate communication lines and storage facilities, consequently it is highly desirable that a large measure of pre-processing be carried out prior to transmission, recording and display.

Instrumentation systems based in the IEEE 488 (HPIB) bus system provide for a flexible instrument control system together with the computing and display capabilities of instruments such as the HP 9835A desktop computer. In this particular system, remote system control can be provided by microwave link if necessary, utilizing a standard RS-232C interface. A further advantage of the HPIB bus system in that a mix of instrument outputs, volts, milli-volts or frequency (vibrating wire instruments), can be accommodated with standard instruments using the same multiplexing unit, which also provides switch closure channels for pre-programmed or random channel calibration.

Further recommendations regarding instrument selection cannot be made without study of the specific installations, calculation of antic-

ipated ranges, desired frequency response, environmental conditions etc. Once this information is established, standard transducers are available which will meet most of the requirements. The most notable exception are the force panels and it is recommended that effort be directed towards the design of large force panels that are capable of measuring the shear force components and locating the point on the surface of which the resultant ice force acts.

11. Instrumentation of Islands

A number of suggestions have been made regarding the use of geographical features such as small islands, steep cliffs, shoals and grounded ice islands as simulated production platforms which could be used to mount ice force measuring equipment.

In considering this possibility the similarity of the ice failure mechanism between island and structure must be considered. An island in shallow water with sloping beaches, will produce a rubble field of broken ice, which rests on the seabed, (Croasdale and Marcellus, 1978). This, in essence, is a totally different failure mechanism to that which would be produced by a monocone or caisson type structure in 60 m of water. If a rubble field were to be produced, it would be unlikely that it would extend downward 60 m to the seabed or provide additional protection for the structure. In all probability, an ungrounded rubble field ahead of the structure would present an increased projected area to the advancing ice and a consequent increase in structural loading.

Therefore in selecting an island which will simulate a structure, there are two possibilities. Ice ridge failure around a conical monopod could be simulated by a small, surface piercing, peak of rock which is

surrounded by deep water. A suitable conical "ridge breaker" could then be mounted over the rock and instrumented in an appropriate manner. A caisson structure might be simulated by a sheer rock face, extending downwards into deep water. The face of the rock could be instrumented with force panels or a string of instrumented barges moored between the advancing ice and the rock face.

The instrumentation of a natural feature would require that it be located sufficiently far offshore to be clear of land fast ice and that there is sufficient ice movement to ensure that large multi-year ridges will be encountered with reasonable frequency.

In assessing the value of such an installation as a simulated structure, it is obviously not going to be possible to study problems related to foundation stability or the detail structural design of the platform. However, the island would represent a rigid feature that would have a very similar effect on the surrounding ice sheet as the structure itself. This fact could be exploited by using the island as a safe location from which to study ridge/structure interaction and to develop the ice instrumentation and monitoring procedures that will eventually be used on the structure itself. The availability of test facilities on such an island would be invaluable for the testing and development of large force panels that might eventually be used on the test structure and for other full scale studies of ice/structure interaction.

12. Experimental Test Structures

The environmental conditions in the Beaufort are such that the greatest potential hazard is the consolidated multi-year ice ridge, there is therefore a need for specific data relating to the

forces that such a ridge can exert on a structure. If modelling studies are carried out on test structures erected in regions subject to only annual ice ridging, then there will always be some uncertainty regarding the validity of the test results when applied to unique Beaufort Sea conditions. Apart from studies intended to gather information on ice/structure interaction, it will almost certainly be necessary to demonstrate that such a structure can survive in the Beaufort Sea for a year or so before full scale production drilling will be permitted. Therefore, a strong argument can be advanced for the immediate construction and evaluation of a full scale pilot structure to be located in the Beaufort Sea.

In view of the enormous cost and construction time for the full scale structure, some consideration should be given to the construction of a 1/5 scale model which could be located in a more accessible region, say in the Gulf of St. Lawrence or Lake Melville. The function of the scale model would be to study the handling and sinking of the structure under real conditions and to perfect methods of deploying the foundation instruments, evaluate various methods of monitoring structural stability, ice interaction studies and generally check out all the operating systems that will eventually be used on the full scale structure.

13. Conclusions and Recommendations

The study of an instrumentation system for an Arctic offshore structure resolves itself, somewhat predictably, into methods of quantitatively defining the ice conditions and measuring the response of the structure to these conditions.

In order to be able to correlate structural response to ice force, an ice tracking and prediction system is required in order to identify

what sections of the ice sheet, together with embedded ridges are likely to impact the structure. Functioning in conjunction with this system would be mobile equipment for ridge drilling, profiling and in-situ strength measurements in ridge depth of up to 35 m.

Direct methods of in-situ (borehole) strength and consolidation measurements may be made using uniaxial borehole jacks, dilatometers and hydraulic fracturing techniques. Various indirect methods such as calibrated drill performance, salinity and temperature measurements are also possibilities. However, these methods are indirect and depend on previously calibrated ice samples. They could also lead to significant errors if used for measurements on unconsolidated ridges.

There is little, if any, published work on the use of hydraulic fracturing techniques for the measurement of ice properties, however, the theory behind this technique has been well developed for rock and it would appear to offer a powerful technique for the in-situ strength measurements of ice, particularly if any degree of porosity or unconsolidation is present. It is therefore recommended that some effort be expanded in the design and evaluation of lightweight mobile drilling equipment capable of carrying out hydraulic fracturing tests under field conditions.

In-situ, ice sheet, stressmeters offer a limited capability of measuring the ice force pushing a ridge, however, they need to be installed well ahead of time in order for creep action to distribute the ice force uniformly over the surface of the transducer.

Wire strainmeters of various types can be installed quickly and would be useful in measuring strains in the surrounding ice sheet as the ridge bears up against the structure. The presence of a stiff ridge changes

the stress distribution in the ice sheet making it difficult to predict where the strainmeters should be located. It is therefore recommended that photoelastic model studies be carried out with the objective of developing guidelines for the placement of strainmeters around a structure.

Expandable tiltmeters installed on the ridge immediately prior to impact would give a measure of ridge motion that could be checked against the theoretical model for ridge clearing forces.

Regarding the instrumentation of the structure, two general techniques are possible, measuring strains within the structure itself or installing discrete force measuring panels on its surface - both techniques should be used in a complementary manner. The detail design of the structure will require the use of finite element techniques which can also be used to identify key locations for the installation of strain gauges. As calibration of the full size structure would be exceedingly difficult, ice force would be measured indirectly by the strain gauges using a calculated value of the structural sensitivity factor. There is a need for an improved design of force panel that can measure shear and adfriction forces in addition to the normal panel force. It is therefore recommended that further study be given to the design and testing of large force panels that would be suitable for use on the full scale structure.

In view of the somewhat indeterminate nature of the seabed reaction, appropriate foundation instrumentation will be an essential part of the study. Geotechnical information taken at the site should be used in a numerical model analysis to identify transducer locations and predict the approximate range of the parameters that are to be measured. Appro-

priate instrumentation can then be selected or designed and deployment techniques worked out.

The instrumentation of a geographical feature such as a small island or rock will not answer all the questions regarding the stability of an offshore structure, however, the availability of such a facility would be invaluable for testing equipment such as force panels, for observation of ice movements for studying the mode of ice/structure interaction and ridge failure.

It is recommended that a 1/5 scale model test of the proposed Beaufort Sea structure be carried out in a relatively accessible area which is subject to severe ice conditions. The primary objective of the exercise would be to study the handling and erection of the structure and to check out the instrumentation and operating systems that will be used on the full scale structure.

REFERENCES

Allan, A. and Winsor, W. (1977) "Industrial Applications of Ice Strain Measurement" POAC - 77 Memmorial Univeristy, St. John's, Newfoundland, pp. 629-637.

Allen, T.O. and Roberts, A.P. (1978) "Production Operations - Well Completions, Workovers and Stimulation" Published by Oil and Gas Consultants International Inc. - Tulsa. V2, p. 148.

Bercha, F.G. and Stenning, D.G. (1979) "Arctic Offshore Deepwater Ice - Structure Interactions" Offshore Technology Conference, OTC 3632 Houston, Texas.

Bourgeois, D. and Leroy, J.P. (1976) "Geophysical and Mechanical Platform Instrumentation" Proc. Conf. on the Design and Construction of Offshore Structures, Institution of Civil Engineers, London.

Browne, R.A. (1975) "The Design of a Strain Gauge Five Component Balance for Measuring Wave Loadings on Models of Offshore Structures" Conference on Measurement in the Offshore Industry organized by BSSM/RINA, Edinburgh, UK.

Coutinho, A. (1949), "Theory of an Experimental Method of Determining Stresses not Requiring an Accurate Knowledge of the Modulus of Elasticity" International Assoc. for Bridge and Structural Engineering, V9, 1949, International Secretariat of Zurich.

Croasdale, K.R. and Marcellus, R.W. (1978), "Ice and Wave Action on Artificial Islands in the Beaufort Sea" Canadian Journal of Civil Engineering V5, March 1978, pp. 98-112.

Danys, J.V. and Bercha, F.G. (1975) "Determination of Ice Forces on a Conical Offshore Structure" POAC - 75 Univ. of Alaska, Fairbanks, pp. 741-751.

Frederking, R. and Gold, L.W. (1971) "Ice Forces on an Isolated Circular Pile" First International Conference on Port and Ocean Engineering Under Arctic Conditions - Norway, V1, 1971, pp. 73-92.

Frederking, R. and Sinha, N.K. (1977) "Ice Action on Wharf at Strathcona Sound" POAC - 77 Memorial University, St. John's, Newfoundland, pp. 707-717.

Haimson, B.C. (1973) "A Simple Method for Estimating In-Situ Stresses at Great Depths" American Society for Testing and Materials - Field Testing and Instrumentation of Rock. STP 55 1973.

Hanna, T.H. (1973) "Foundation Instrumentation" published by Trans Tech. Publications 1973, p. 140.

Hnatiuk, J., Kovacs, A. and Mellor (1978 "A Study of Several Pressure Ridges and Ice Islands in the Canadian Beaufort Sea, Journal of Glaciology, V20, No. 84, 1978, pp. 519-532.

Hobbs, R., George, P.J. and Mustone, G.G.W. (1978) "Some Applications of Numerical Methods to the Design of Offshore Gravity Structure Foundations" Numerical Methods in Offshore Engineering, ED. Zienkiewiez OC et al, Published by Wiley.

Howard, G.C. and Fast, C.R. (1970) "Hydraulic Fracturing", Published by Society of Petroleum Engineers of AIME, p. 11.

Jazrawi, W. and Khanna, J. (1977) "Monocone - A Mobile Gravity Platform for the Arctic Offshore POAC - 77 Memorial Univeristy, St. John's, Newfoundland, pp. 170-184.

Ladanyi, B. (1979) "Borehole Relaxation Tests as a Means for Determining the Creep Properties of Ice Covers" POAC '79, Trondheim, p. 757.

Maattanen, M. (1977) "Ice-Force Measurements at the Gulf of Bothnia by the Instrumented Kemi I Lighthouse" POAC - 77 Memorial University, St. John's, Newfoundland, pp. 730-740.

Metge, M., Strilohu,, A. and Trofimenkoff, P. (1975) "On Recording Stresses in Ice" IAHR Symposium on Ice Problems, Hanover, New Hampshire, pp. 459-468.

Michel, B. (1978) "Ice Mechanics", Published by Les Presses de L'Universite' Laval, Quebec, p. 99.

Neill, C.R. (1970) "Ice Pressure on Bridge Piers in Alberta" IAHR Symposium - Ice and its Action on Hydraulic Structures Reykjavik, Iceland.

Nelson, R.D. (1975) "Internal Stress Measurements in Ice Sheets Using Embedded Load Cells" POAC - 75 University of Alaska, Fairbanks, pp. 361-374.

Nelson, R.D., Taurianen, M. and Borghorst (1976) "Techniques for Measuring Stress in Sea Ice" Alaska Sea Grant Report 76-18, University of Alaska, Fairbanks.

Prange, B. (1972) "The State of Telemetry in Soil Mechanics" Roscoe Memorial Symposium - Cambridge Editor Foulis, pp. 476-488.

Ralston, T.D. (1977) "Ice Force Design Considerations for Conical Offshore Structures" POAC - 77 Memorial University, St. John's, Newfoundland, pp. 741-752.

Rocha, M. (1969) "New Techniques in Deformability Testing of In-Situ Rock Masses" American Society for Testing and Materials - Determination of the In-Situ Modulus of Deformation of Rock STP 477, 1969.

Sackinger, W.M. and Nelson, R.D. (1979) "Sea Ice Stresses Near Grounded Obstacles" Offshore Technology Conference OTC 3528 Houston, Texas.

Sackinger, W.M. and Sackinger, P.A. (1977) "Shear Strength of the Adfreeze Bond of Sea Ice to Structures" POAC - 77 Memorial University, St. John's, Newfoundland, pp. 607-614.

Stenning, D.G. and Schumann, C.G. (1979) "Arctic Offshore Deepwater Ice-Structure Interaction" Offshore Technology Conference, OTC 3630 Houston, Texas.

Templeton, J.S. (1979) "Measurement of Sea Ice Pressures" POAC '79, Trondheim, p. 73.

Timoshenko, S. and Goodier, J.N. (1951) "Theory of Elasticity", Pub. McGraw Hill, pp. 85-107.

Wright, B., Hnatiuk, J. and Kovacs, A. (1979) "Multi-year Pressure Ridges in the Canadian Beaufort Sea" POAC '79, Trondheim, p. 107.

A SELECTION OF ISLANDS
FOR USE AS A TEST PLATFORM

by

Chantal Roche

Centre for Cold Ocean Resources Engineering

Memorial University of Newfoundland

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1.0 Introduction

The destructive potential that a moving ice cover presents to an offshore structure is of vital concern in the design of offshore facilities for use in northern waters. This report documents a search to identify potential test sites for a study of the forces exerted by one of the most formidable hazards contained in a mobile ice cover — multi-year pressure ridges. Ridges with sails as high as 13 m and keel depths of 47 m have been reported in the Arctic (Weeks, 1976). It has been suggested that a structure designed to withstand the force exerted by a "100 year ridge", having a draft of over 45 m, could be considered capable of safe operation amidst any moving ice cover.

Two approaches are being considered to measure these forces. The initial approach considers the feasibility of using a small island or rock surrounded by a suitable ice cover as a platform for instrumentation systems. The second approach envisages the construction of small scale model or tower that could be instrumented to measure the forces exerted by the surrounding ice cover. To provide a foundation for the tower, a natural geological feature such as a submerged rock or shoal would be used.

The concept of using a small island has been employed successfully in the past for long-term observation of ice movement. A C-CORE pilot project in March, 1979 observed and recorded the interaction between pack ice drifting southward from the Labrador Sea against a small island of the east coast of Newfoundland. A time-lapse camera and inexpensive marine radar system was installed on the island to obtain a continuous record of the ice movement around the island. However, the ice condi-

tions that were observed around this island (small to medium floes of predominately first-year ice) would not provide a representative measure of the conditions likely to be faced by drilling platforms in the Beaufort Sea. A search was initiated to find a number of small islands or rocks more suitable for use as an instrument platform.

1.1 Procedure

After the characteristics of the ideal test site had been established the search began with the selection of particular channels or coastlines that most closely resembled the ideal in terms of ice age and concentrations. The charts of these areas were then examined to locate any offshore rocks or islands that could be considered as potential test platforms.

The sites selected have been grouped into three sections:

(1) Beaufort Sea and Arctic Ocean

(2) Canadian Arctic Archipelago

(3) Eastern Arctic

1.2 Selection Criteria

The ideal platform could be described as a steep sided geologic feature of a size comparable to an offshore rig, located in an area of dynamic pack with a high component of multi-year ice. Water depths should allow for the possibility of an impact with a "100-year pressure ridge" mentioned previously. With this idea in mind, the following broad characteristics were identified as selection criteria for the test site:

1. Ice Cover Characteristics

   a) greater than ten percent of total ice cover should consist of second-year or multi-year ice.

   b) ice cover immediately surrounding the site should be dynamic to allow for the formation of large pressure ridges or for ridges formed elsewhere to drift into the area.

2. Platform Characteristics

   a) to minimize scaling effects the site should have a maximum dimension under 1.6 kilometres.

   b) a sheer sided island or rock would eliminate bathymetric effects which are likely to cause grounding of floes or the creation of a rubble field. Deep water (greater than 50 metres) surrounding the site would also allow the impact of large ridges to take place.

1.3 Data Sources

The principle sources consulted in the search were:

(a) Various navigational charts published by the Canadian Hydrographic Service, Department of Energy, Mines and Resources, Ottawa.

(b) The Pilots and Sailing Directions published by the Canadian Hydrographic Service, Department of Energy, Mines and Resources Ottawa and the Department of Fisheries and the Environment, Ottawa.

(c) Various topographic maps published by Surveys and Mapping, Department of Energy, Mines and Resources, Ottawa.

(d) Various aerial photographs held at the National Air Photo Library, Department of Energy, Mines and Resources, Ottawa.

(e) An Arctic Atlas: Background Information for Developing Marine Oilspill Countermeasures produced by FENCO Consultants Ltd. and F.F. Slaney & Company Ltd., Arctic Marine Oilspill Program Report EPS-9-EC-78-1 published by Environmental Impact Control Directorate, Fisheries and Environment Canada, Ottawa.

(f) Various other published reports documenting ice surveys or research carried out under both government and industry sponsorship.

(g) Personal communications with several persons, particularly mariners who have spent time working in the study area.

2.0 Results

While many of the northern coastal waters are fringed with innumerable rocks, islands and shoals there is a marked scarcity of the detailed information required to evaluate their potential for use as a test platform.

2.1 Ice Conditions

A large data base was available that enabled an assessment of general ice conditions on a regional basis to be made. The Arctic Atlas was most useful in locating areas with the required concentrations of multi-year ice (Figures 1 and 2). It provided valuable summaries of ice conditions compiled from historical records of the Atmospheric Environment Service, the U.S. Hydrographic Office, Polar Continental Shelf Data and industry records.

The general summaries and notes on ice conditions contained in the Pilot were particularly helpful in establishing the level of motion in the ice cover.

More site specific information was available from ice survey reports and from the personal experiences of individuals contacted.

2.2 Water Depths

The navigational charts and Pilots provided the sole sources of data on water depths. With many of the channels in the Arctic Regions yet to be surveyed, it was not unexpected that very few soundings were available along the coastlines considered in the search. A large proportion of the charted depths for these areas were obtained from

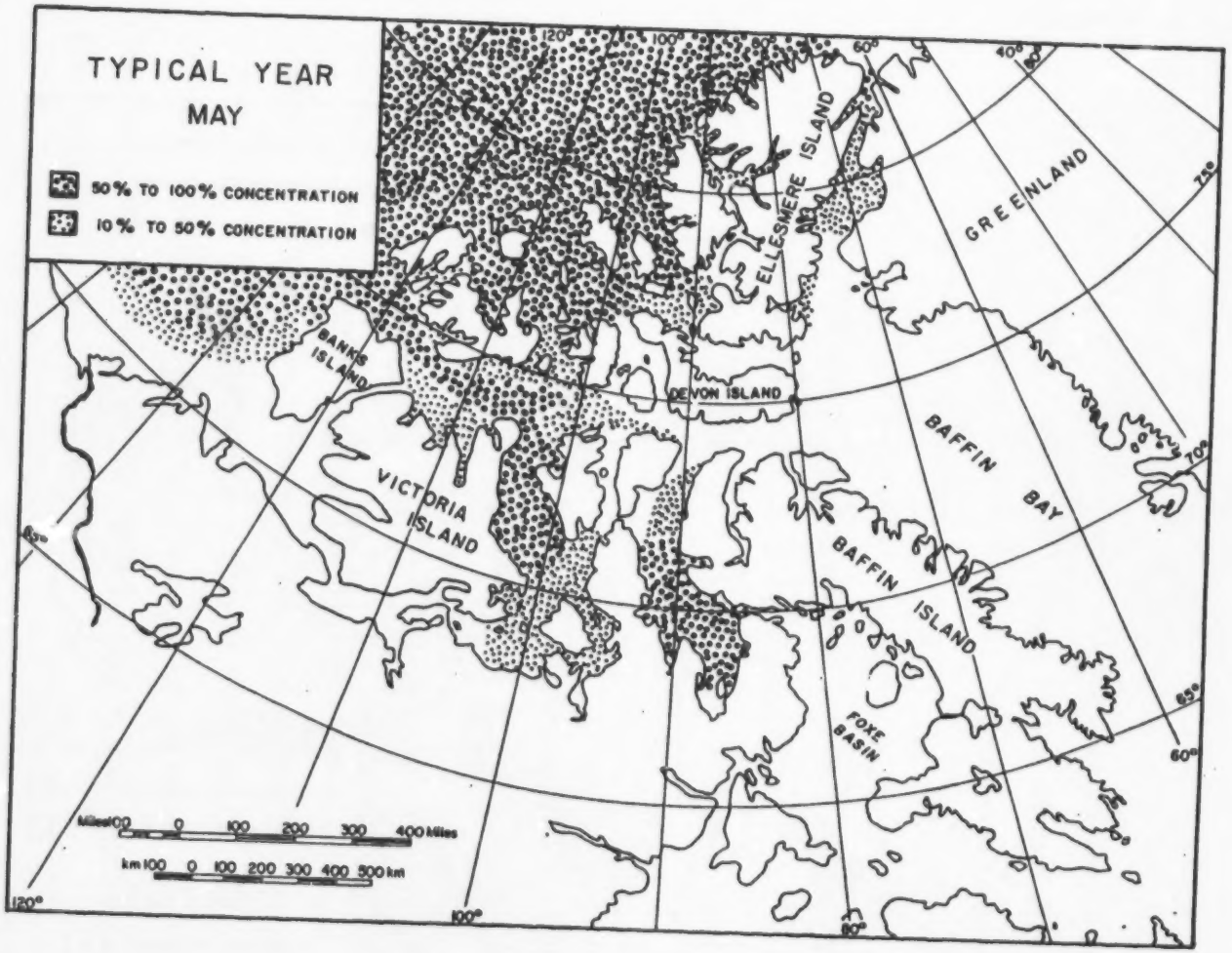

Figure 1: General summary of the distribution of second year and older ice in May of a typical year.

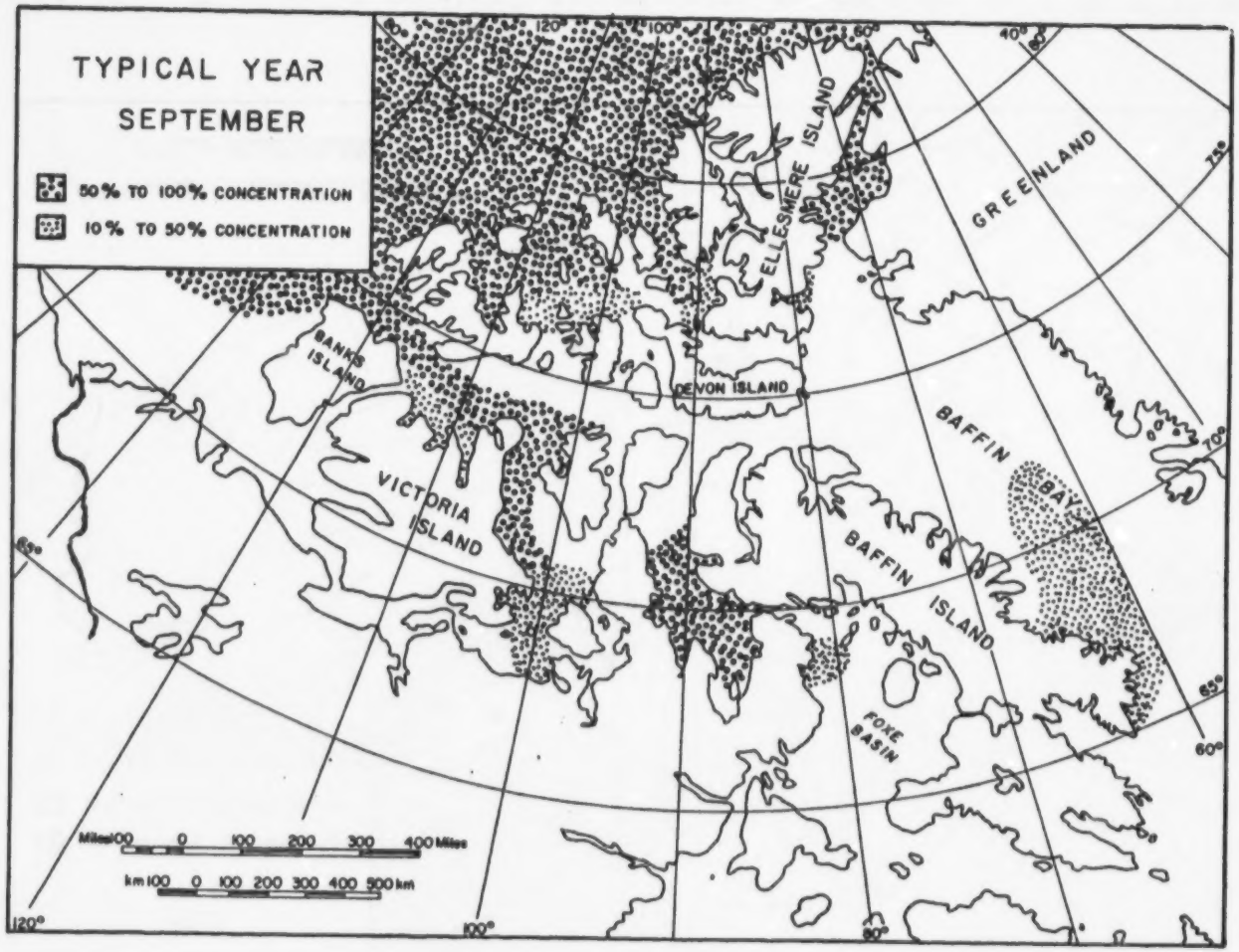

Figure 2: General summary of the distribution of second year and older ice in September of a typical year.

individual ship track soundings. The Pilot cautions that while accurate as to depths, these soundings may be misleading in their charted positions.

2.3 Size and Shape

There were no published maps or charts found of the Arctic Regions that contained sufficient topographic detail to evaluate the size or shape of the small islands of interest. The largest scale topographic maps available, with a few exceptions, were from the 1:250,000 series. Arctic charts were mostly 1:500,000 scale, or at best 1:200,000. Spot elevations were noted for a few of the larger islands but generally there was a complete absence of topographic detail.

Much of the data presented on size and shape has been extracted from the Pilot of Arctic Canada or estimated from the small scale maps that were available.

Since the available maps of the Arctic regions are largely based on aerial photographs, the logical source for more detailed information was to return to this original record. Vertical airphotos, taken from an altitude of 9100 m with either a 152 mm or 88 mm lens are available for all land areas. However, the coverage frequently does not include mid-channel or offshore areas. Snow cover and ice concentrations generally greater than 5/10 also limited the utility of the airphotos, particularly for small islands and rocks. The orientation of the flight lines in particular made the search of the airphotos held at the National Airphoto Library extremely time consuming.

2.4 Beaufort Sea and Arctic Ocean

The coastal waters bordering the zone of permanent polar pack ice were among the first to be considered in the search.

The islands along the continental coast of the Beaufort Sea lie within the shallow coastal shelf that in places extends 80 to 100 km offshore. Any large multi-year floes and pressure ridges contained in the pack ice driven onshore are likely to ground on the extensive shoals found along the coast. Consequently, none of these islands were recommended as test sites.

Nevertheless, there is one feature of this coastline which deserves a mention. Hydrographic surveys indicate the possible existence of pingo-like features between latitudes $70^{o}10'N$ and $71^{o}10'N$ and longitudes $130^{o}35'W$. Water depths in this area fall between 30-60 m. It may be feasible to use one of these submerged dome-shaped knolls as a foundation for a test structure.

Four groups of islands were identified as potential test sites, all with year-round exposure to the polar pack ice. The available information on the four groups is summarized in Tables 1 to 4.

Table 1

Gore Islands Latitude 74°19'N Longitude 124°58'W

Chart No. 7832 shown in Figure 3

General Description: Two islands and one islet lying WSW of Cape Prince Alfred off the western Coast of Banks Island. Two submerged rocks with 1.8 m over them lie 1.6 km WSW + 3 km SW of the outer Gore Island.

Water Depths: This area lies within the 10 m isobath, however the bottom shelves rapidly to depths of over 200 m to the north of the islands. Between the two islands depths of 16.5 m and 27.4 m have been reported.

A shoal lies 13.6 km SW of the outer island.

Surface Currents: The direction is dependent on tidal cycles and wind conditions. There is evidence of a prevailing northerly set along most of the west coast of Banks Island (Oceanographic Services, 1969).

Ice Conditions: General ice summaries indicate that concentrations of between 50 and 100% multi-year ice year-round are typical. During the summer months varying amounts of landfast ice and loose ice are always present with the polar pack never far offshore. Onshore movement of the pack in response to offshore rather than onshore winds is a frequent occurrence.

Size and Shape: (See air photo in Figure 4)

The islet to the north of the Gore Islands and the submerged rocks deserve further investigation as potential test platforms. The islet is less than .5 km long and regular in shape. It appears to be low lying although no estimation of height is available.

Neither of the Gore Islands match the selection criteria in size or shape. The larger Gore Island follows an irregular S-shaped outline averaging less than .5 km in width but exceeding 3.5 km in length. An elevation of 43 m is charted in the centre of the island. The outer Gore Island is smaller (approximately .8 km x 1.8 km) and is roughly triangular in outline. Elevations are between 15 and 30 m.

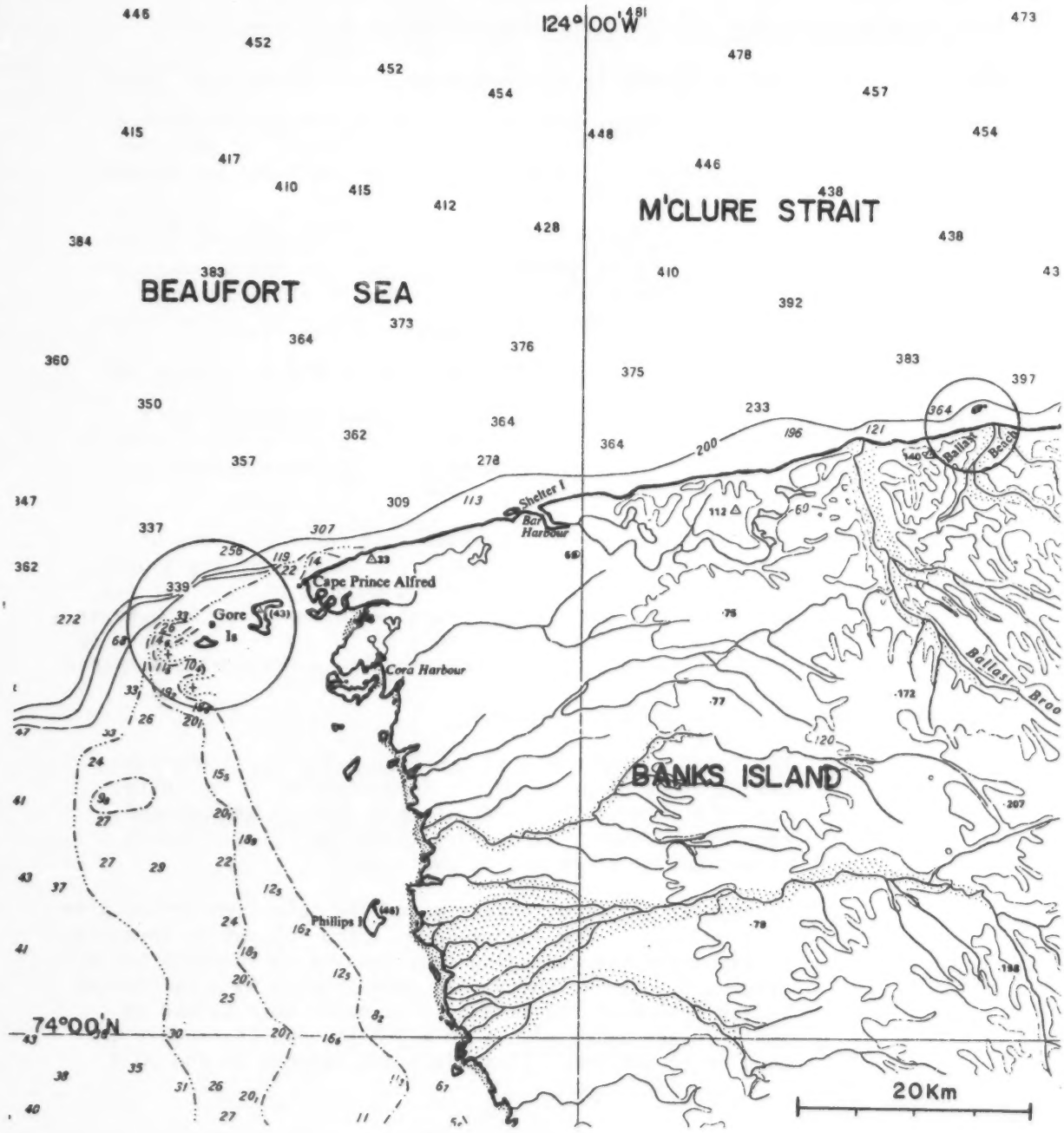

Figure 3: Gore Island and Unnamed Islands off Ballast Beach, Banks Island. Portion of Chart No. 7832. 1971 edition. Original scale 1:500,000. Depths in metres.

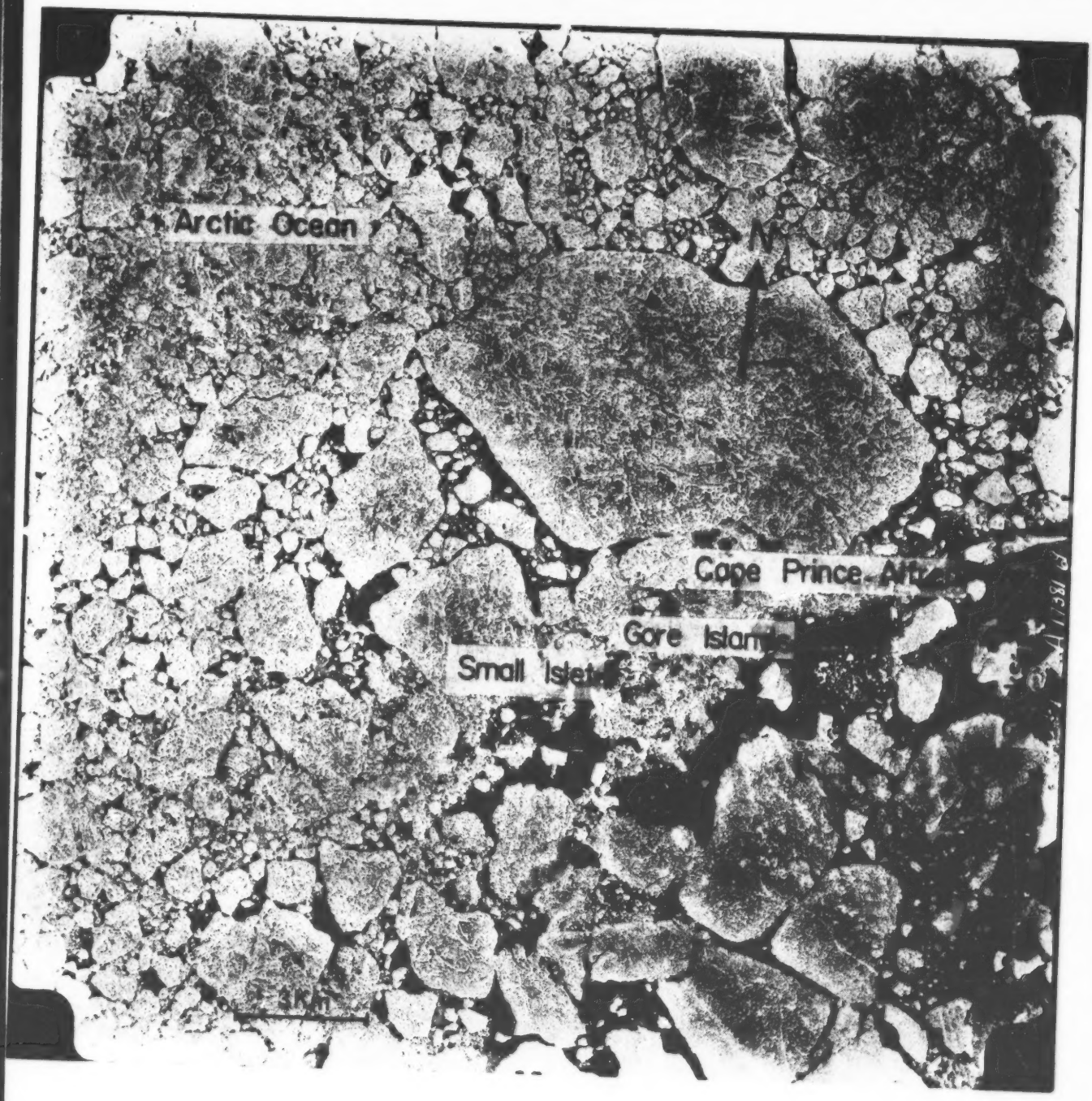

Figure 4: Gore Islands Aerial photograph taken from an altitude of 9100 m with a 88 mm lens. August, 1961. Original photo supplied by the Surveys and Mapping Branch, Department of Energy, Mines and Resources.

Table 2

Polynia Islands Latitude 77°40'N Longitude 115°53'W

Chart No. 7951 shown in Figure 5

General Description: Two islands and numerous small islets lying NE of Cape Leopold M'Clintock off the north coast of Prince Patrick Island. The Polynia Islands form the above water parts of a shoal that lies across the NW entrance to Ballantyne Strait.

Water Depths: The waters around the Polynia Islands are extremely shallow with well developed sandbars. Charted depths outside the sandbars range from 27 to 97 m. Less than 20 m of water lies immediately off most of the islets but water depths between the groups of islands are well over 50 m.

Surface Currents: The general surface flow through Ballantyne Strait is southeasterly.

Ice Conditions: The islands provide a barrier to the movement of ice from the Arctic Ocean through the Strait. Greater than 50% concentrations of multi-year ice are typical year-round.

Size and Shape: (See air photo in Figures 6 and 7)

The Polynia Islands are narrow, low lying and about 6 km in length. The northeastern island is 17 m high and the southwestern island 26 m high. No details are available on the islets but they average less than .5 km in size.

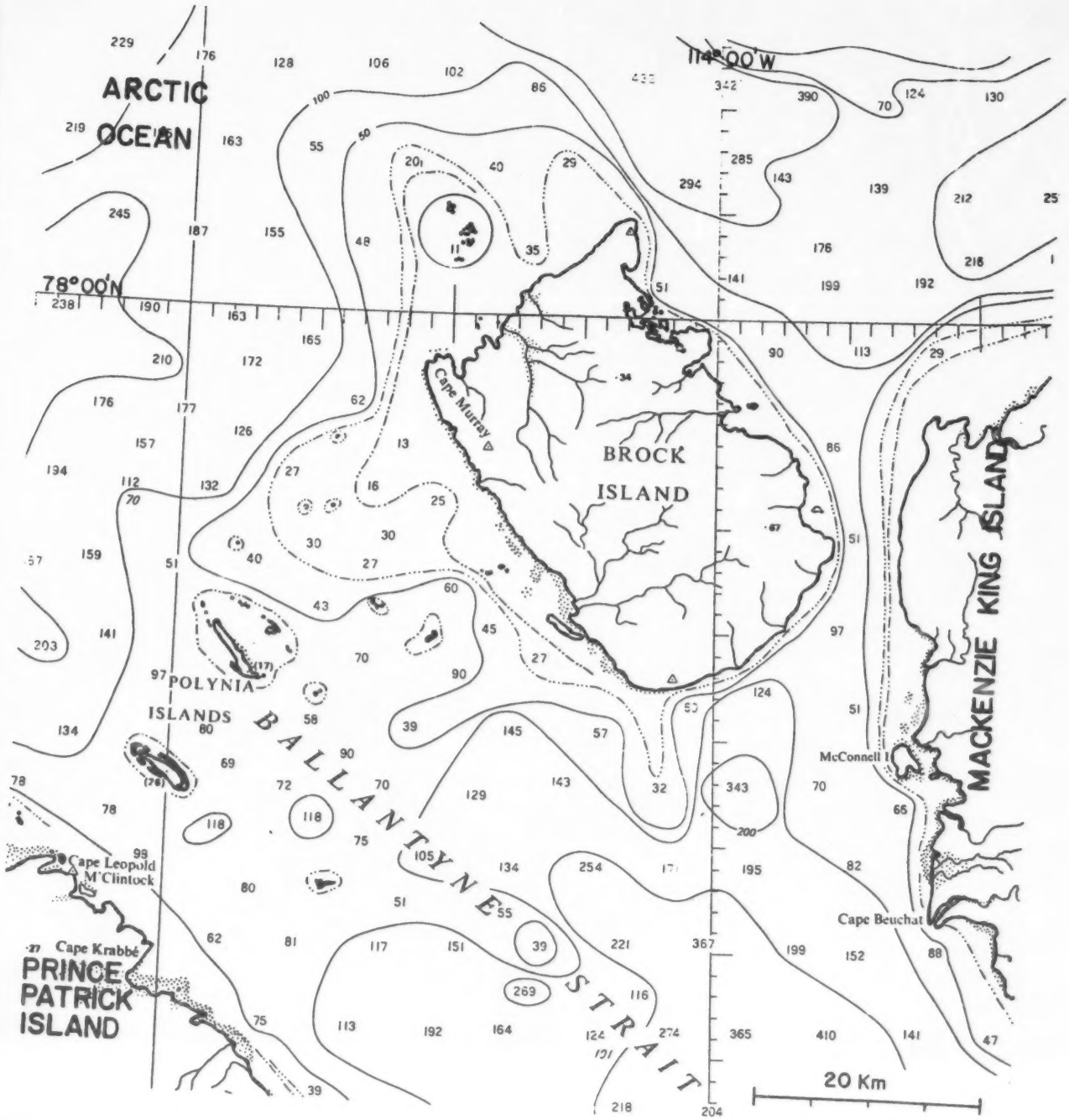

Figure 5: Polynia Islands and Unnamed Islands north of Brock Island. Portion of Chart No. 7951, 1971 edition. Original scale 1:500,000; Depths in metres.

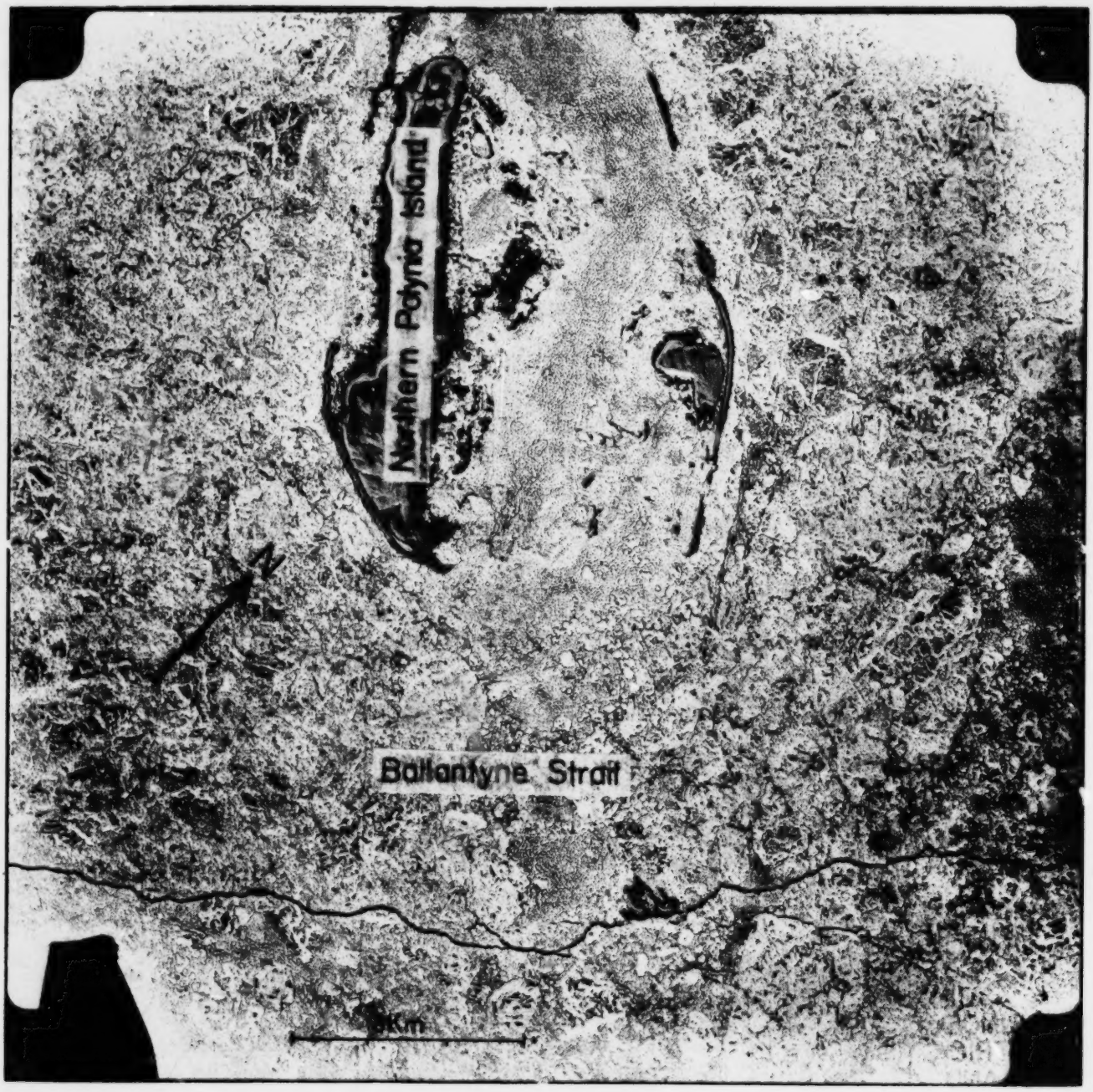

Figure 6: Polynia Islands Aerial photograph of the North-eastern Polynia Island taken from an altitude of 9100 m with a 88 mm lens. August, 1960. Original photograph supplied by the Surveys and Mapping Branch, Department of Energy, Mines and Resources.

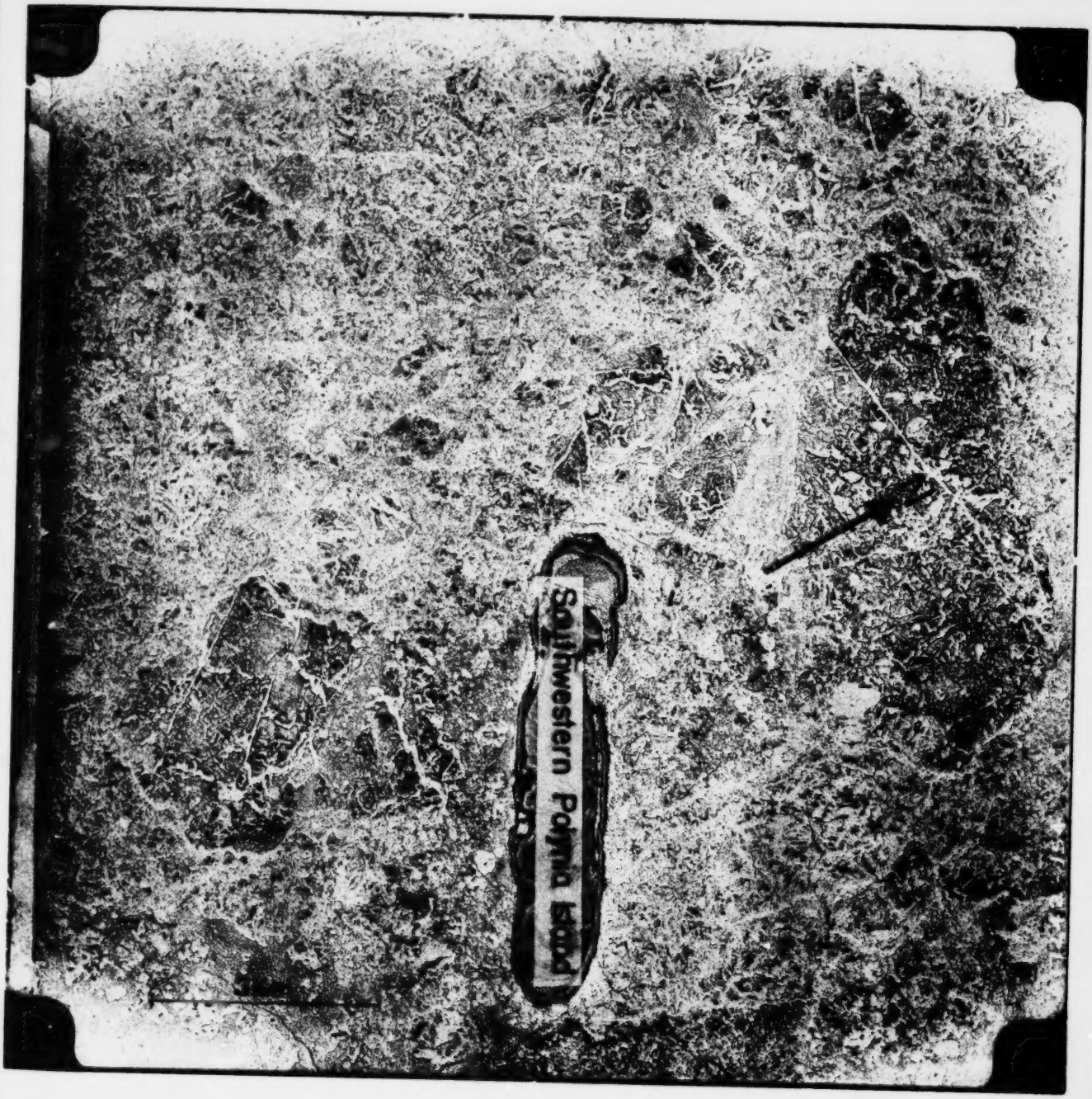

Figure 7: Polynia Islands Aerial photograph of the Southwestern Polynia Island taken from an altitude of 9100 m with a 88 mm lens. August, 1960. Original photograph supplied by the Surveys and Mapping Branch, Department of Energy, Mines, and Resources.

Table 3

Group of Islands north of Brock Island Latitude 78°04'N Longitude 115°00'W

Chart No. 7951 shown in Figure 5

General Description: Three groups of closely spaced islets situated 10 km north of Cape Murray.

Water Depths: Isobaths on the chart indicate general depths of less than 20 m. An isolated sounding of 11 m is shown between the two most southerly groups.

Surface Currents: The islets lie on the edge of the southwesterly drift of the Arctic Ocean.

Ice Conditions: A permanent ice cover of 50 and 100% multi-year ice is indicated on general ice summaries. The shear zone between the landfast ice bordering the Queen Elizabeth Islands and the polar pack would be expected to lie very close to these islets.

Size and Shape: (See air photo in Figure 8)

The islets are all low lying and very irregular in shape. Individually their dimensions fall under .5 km.

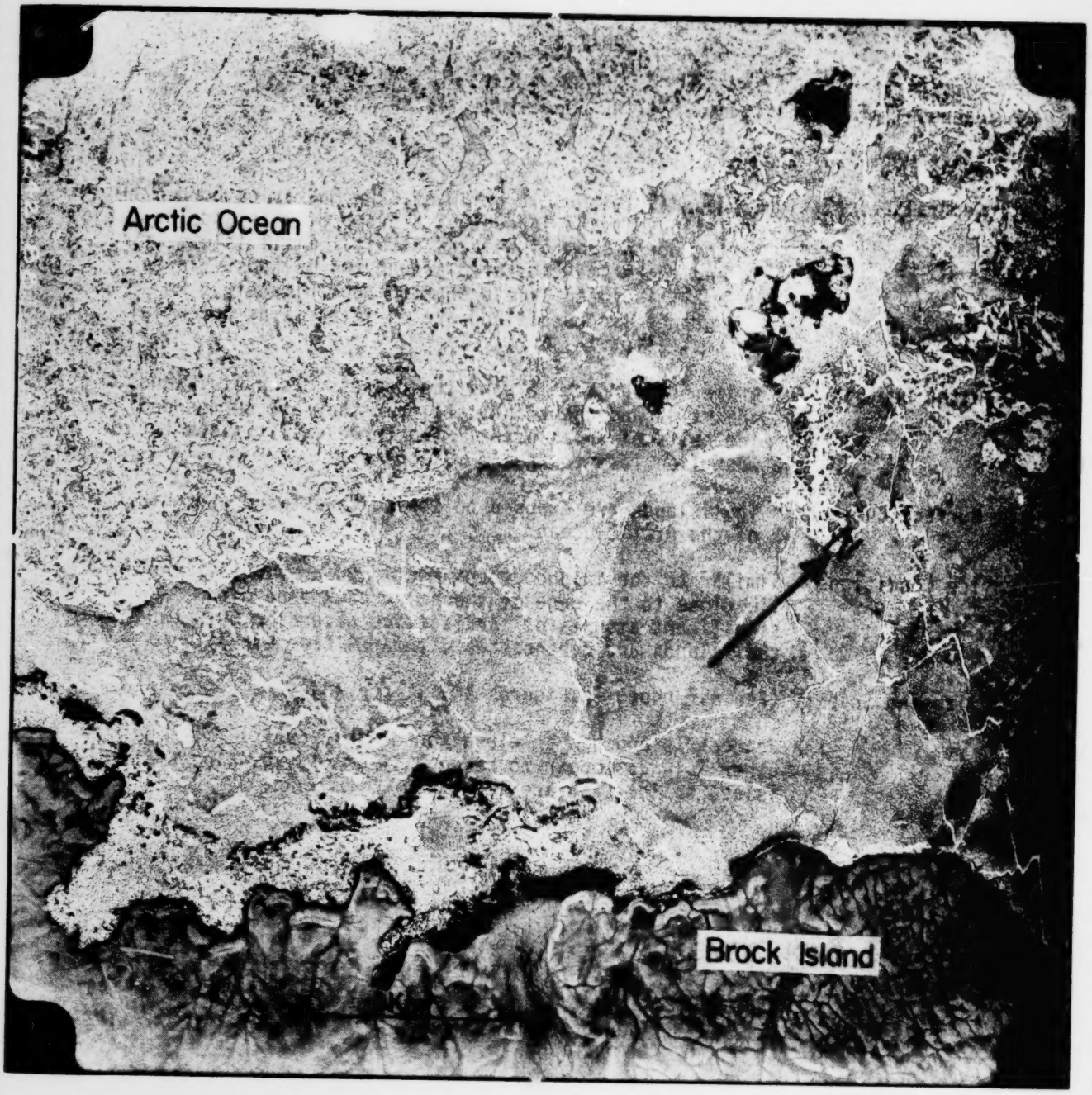

Figure 8: Group of Islands north of Brock Island. Aerial photograph taken from an altitude of 9100 m with a 88m lens, August, 1960. Original photograph supplied by the Surveys and Mapping Branch, Department of Energy, Mines and Resources.

Table 4

Jenness Island Latitude 78°17'N Longtitude 113°55'W

Chart No. 7951 shown in Figure 9

General Description: Jenness Island, 15 km SW of Cape Mackay on Borden Island, is the largest of a group of 8 islands on the north side of Wilkins Strait. Small islets and rocks are clustered around the islands. Three small isolated islets charted approximately 5 km west of Jenness Island are of interest as potential test sites.

Water Depths: There are no soundings around the Jenness Group of islands, but the Pilot and chart indicate that they are surrounded by wide belts of shoal water. The three small islets to the west lie close to the 30 m isobath.

Surface Currents: The islands are exposed to the southwestward currents of the Arctic Ocean.

Ice Conditions: Similar to the island groups mentioned previously, they lie close to the edge of the polar pack zone with significant amounts of landfast ice occurring between the islands and northeastward towards Cape Mackay.

Size and Shape: (See air photo in Figures 10, 11 and 12)

Jenness Island has an elevation of just over 12 m. The other islands appear to be equally low lying. The small islets to the west are less than .5 km long.

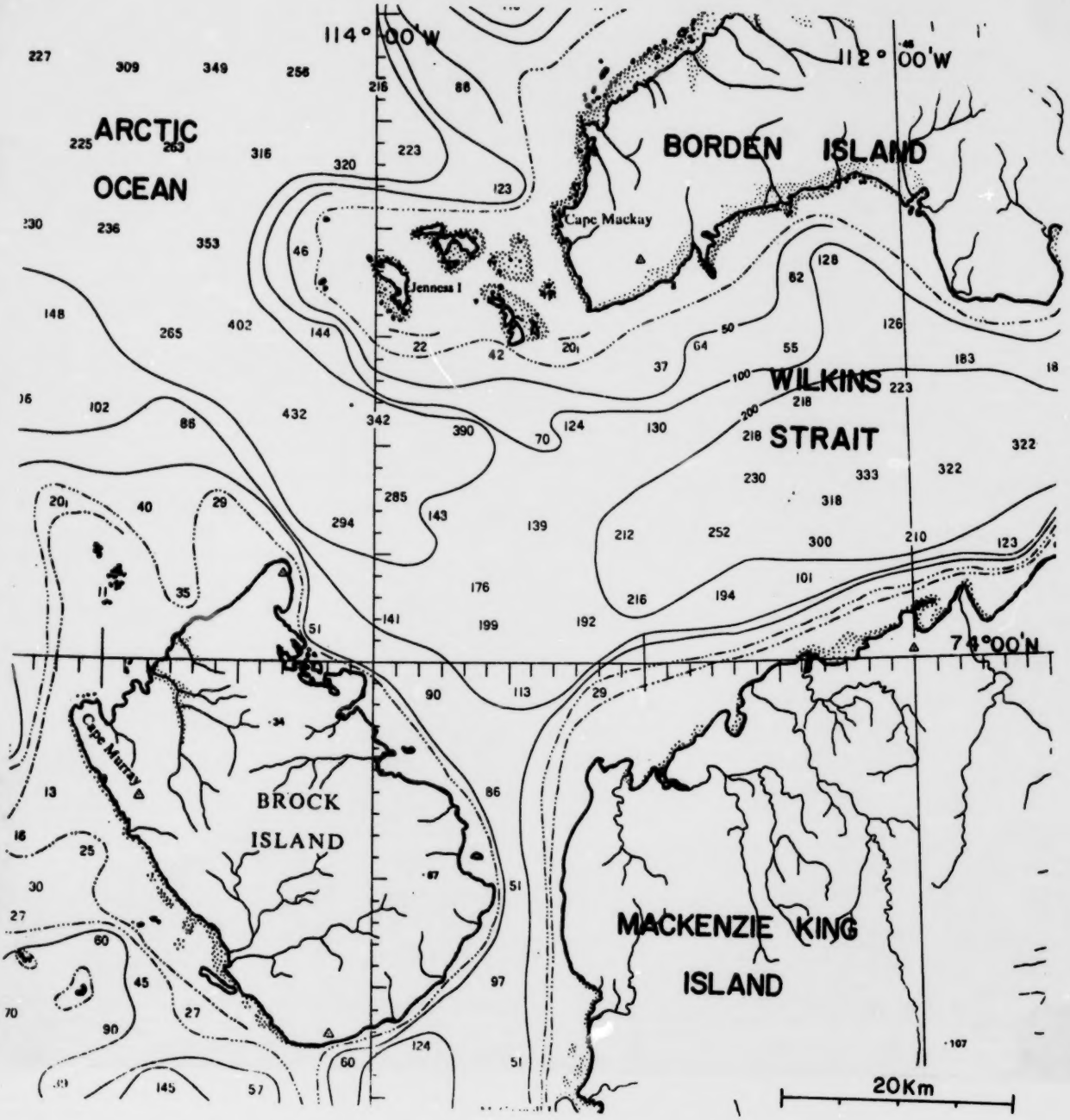

Figure 9: Jenness Island Portion of Chart No. 7951, 1971 edition. Original scale 1:500,000. Depths in metres

Figure 10: Jenness Island Aerial photograph near Jenness Island taken from an altitude of 9100 m with a 88 mm lens, August, 1960. Original photograph supplied by the Surveys and Mapping Branch, Department of Energy, Mines and Resources.

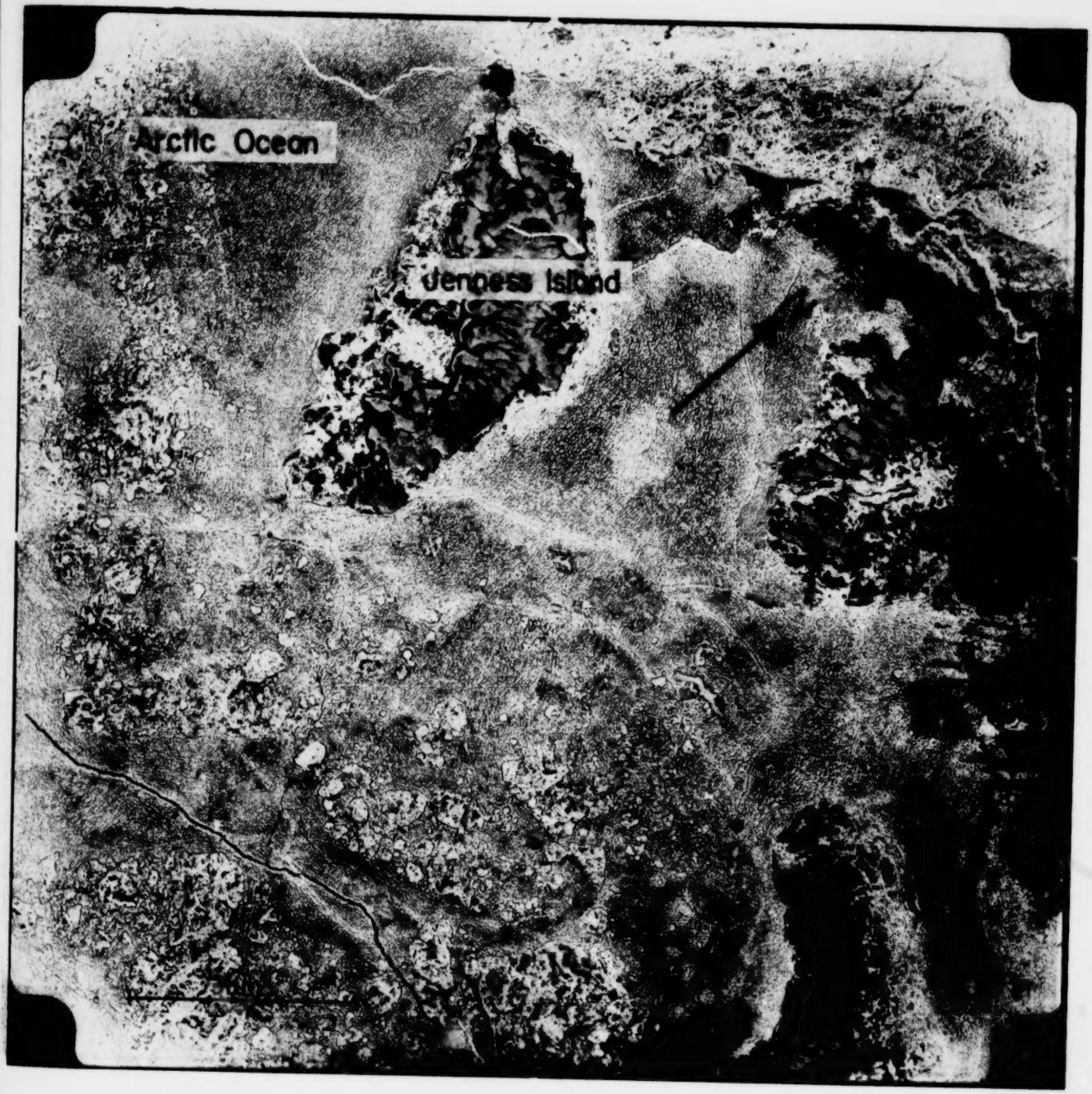

Figure 11: Jenness Island. Aerial photograph of Jenness Island taken from an altitude of 9100 m with a 88 mm lens, August, 1960. Original photograph supplied by the Surveys and Mapping Branch, Department of Energy, Mines and Resources.

Figure 12: Jenness Island Aerial photograph of the islands between Jenness Island and Cape Mackay taken from an altitude of 9100 m with a 88mm lens, August, 1960. Original photograph supplied by the Surveys and Mapping Branch, Department of Energy, Mines and Resources.

2.5 The Canadian Arctic Archipelago

Islands situated in the channels north of Melville and Bathurst Islands were not included in the search for a test site due to unsuitable ice conditions. Although more than 50% of the ice cover is made up of multi-year ice, overall movement is severely limited, typically less than 3 metres per day.

Similarly the southern channels along the continental coast and the channels and bays near Eureka Sound did not meet the ice criteria. The predominately first-year ice cover remains immobile during the winter months and clears from the area after breakup.

Among the remaining channels in the archipelago, annual variations in both the degree of breakup and the extent of clearing or influx of multi-year ice can vary considerably from year to year. M'Clure Strait and Viscount Melville Sound frequently remain choked with ice year round but may open up considerably in a light ice year. The eastern end of Parry Channel normally is cleared of its first-year ice cover in summer.

The southward drift of second year and older ice through M'Clintock Channel, the Gulf of Boothia and Committee Bay identified these areas as particularly promising. Unfortunately a scarcity of data on the coastal waters rapidly eliminated them from further consideration.

The five potential test sites chosen for consideration are near the major transportation routes through the archipelago, consequently more data are generally available. Tables 5 to 9 outline the five sites.

Table 5

Hamilton Island Latitude 74°08'N Longitude 99°22'W

Chart No. 7829 shown in Figure 13

General Description: The smallest of three islands located in mid-channel of Barrow Strait. It lies 26 km north of Cape Grey on Russell Island.

Water Depths: Shoal water extends along the northern side of the island with more extensive shoals reported to the northeastern side. Soundings of over 100 m to the south of the island are indicative of deeper waters along this coast.

Surface Currents: An eastward flow predominates on this side of Barrow Strait. Maximum current speeds of 150 cm/s have been reported (Fenco and Slaney, 1979).

Ice Conditions: The ice cover is usually stationary from December to July. An average of 15% old ice is typical during the winter. The movement of ice through Barrow Strait is restricted by the presence of the mid-channel islands consequently up to 5/10 cover may remain throughout the summer.

Size and Shape: The northside of Hamilton Island is low lying while a maximum elevation of 23 m is found along the southside. Approximate dimension of the island are 1.6 km north-south and .7 km east-west.

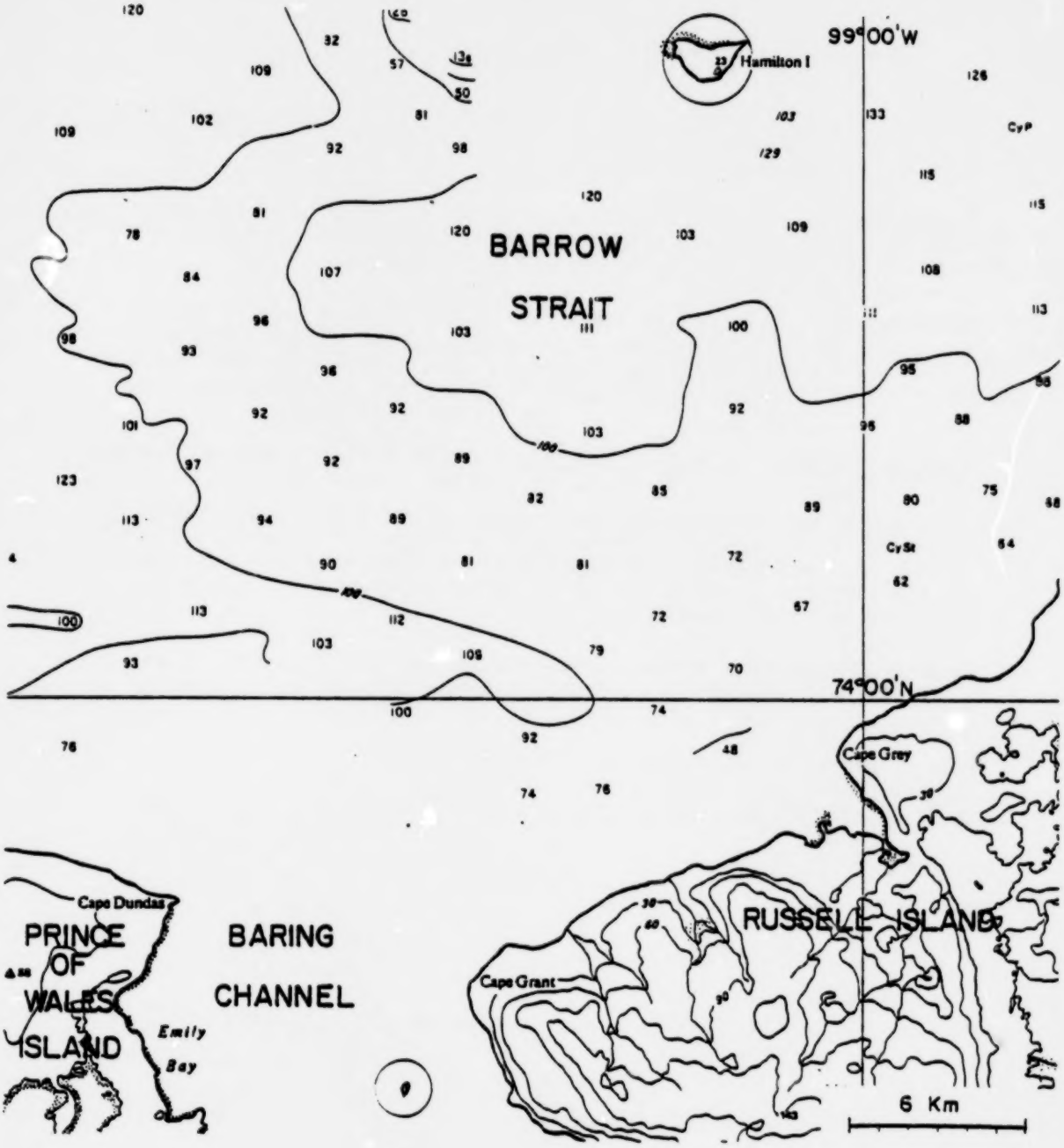

Figure 13: Hamilton Island and Unnamed Island in Baring Channel. Portion of Chart No. 7829, 1977 edition. Original scale 1:200,000; Depths in metres

Table 6

Unnamed Island in Baring Channel Latitude 73°53'N Longitude 99°30'W

Chart No. 7829 shown in Figure 13

General Description: A small island lying in the western entrance to Baring Channel approximately 3.5 km SW of Cape Grant on Russell Island.

Water Depths: There are no soundings in the vicinity of the island. A single track of soundings in the eastern section of Baring Channel indicates deep water. Charted, depths in Barrow Strait north of the island are greater than 50 metres.

Surface Currents: No data available.

Ice Conditions: Although Baring Channel can be cleared each year, heavy ice is thought to remain at the western entrance. In September of a typical year concentrations in excess of 50% of multi-year ice are found in this area.

Size and Shape: The island is less than 1 km long and under .5 km wide. No estimate of the height of the island is available.

Table 7

Unnamed Islands in Hadley Bay Latitude 72°53'N Longitude 109°00'W

Chart No. 7831 shown in Figure 14

General Description: The islands of interest are situated in the western entrance to Hadley Bay. They form two small groups lying 3 and 7 km respectively NE of Wilfred Brown Island.

Water Depths: Soundings from 65 to 295 m are found in Viscount Melville Sound at the head of Hadley Bay. There are no water depths recorded in Hadley Bay.

Surface Currents: A general eastward flow occurs in Viscount Melville Sound. No data is available for Hadley Bay.

Ice Conditions: The disintegration of locally formed ice in Hadley Bay may leave the southern reaches of the Bay open during the summer months. Ice conditions in the northern portion of the Bay depend upon the degree of breakup in Viscount Melville Sound. Between mid-August and the end of October a mobile ice cover of predominately multi-year ice may be driven by wind into the entrance of Hadley Bay from the Sound.

Size and Shape: The islands of interest appear to be less than .5 km in size. Typical elevations for the islands in this area of Hadley Bay are reported to be from 30 to 60 m in the Arctic Pilot.

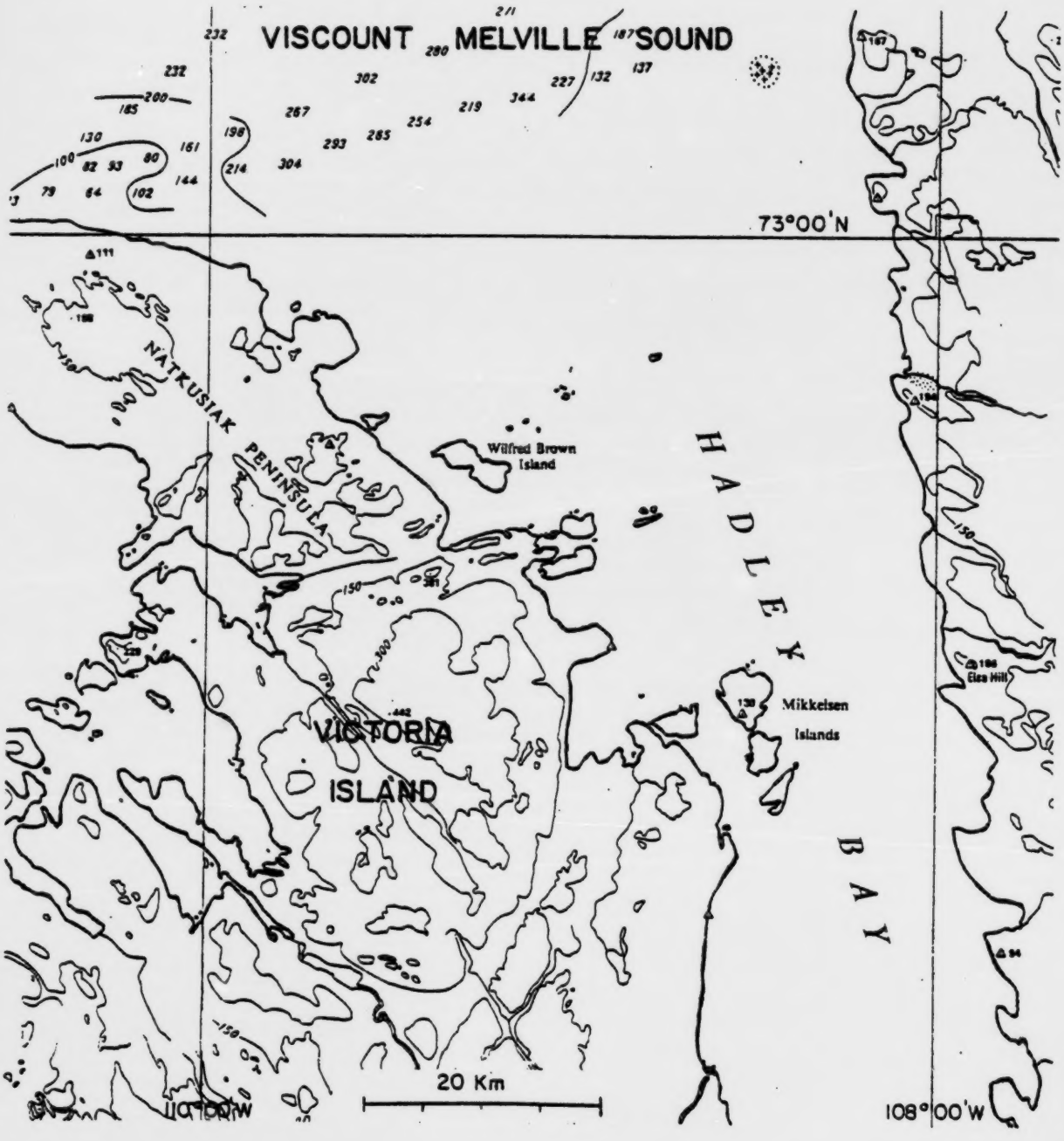

Figure 14: Unnamed Islands in Hadley Bay. Portion of Chart No. 7831, 1976 edition. Original Scale 1:500,000; Depths in metres.

Table 8

Princess Royal Islands Latitude 72°46'N Longitude 118°03'W

Chart No. 7833 shown in Figure 15

General Description: Two isolated islands in the centre of Prince of Wales Strait. The larger of the two lies 27 km west of Johnson Pt. on Banks Island.

Water Depths: Extensive shoals with water well under 20 metres lie between the islands and Banks Island to the west. Deeper water ranging from 50 to over 100 metres is found to the east of the islands. Three isolated shoals are charted to the northeast. The water between the islands ranges from 27 to .4 metres in depth.

Surface Currents: The Arctic Pilot describes the prevailing currents as northeasterly along the eastern side of the Strait and southwesterly on the western side. Tidal currents of up to 2 knots (1 m/sec) experienced in September 1970 by the Survey Ship "Baffin" are also reported.

Ice Conditions: First-year ice forms a solid cover over the Prince of Wales Strait from late October until late July. Clearing of all but the northern portions of the Strait are typical during August and September. Only rarely does multi-year ice from Viscount Melville Sound find its way into the Strait driven by northeasterly winds.

Size and Shape: (See photographs in Figures 16 and 17)

The smaller of the two islands (1 km x 46 m) has an elevation of 19 metres. The island to the south is less than .5 km wide but almost 2 km in its north-south dimension. A height of 58 metres has been surveyed at its highest point.

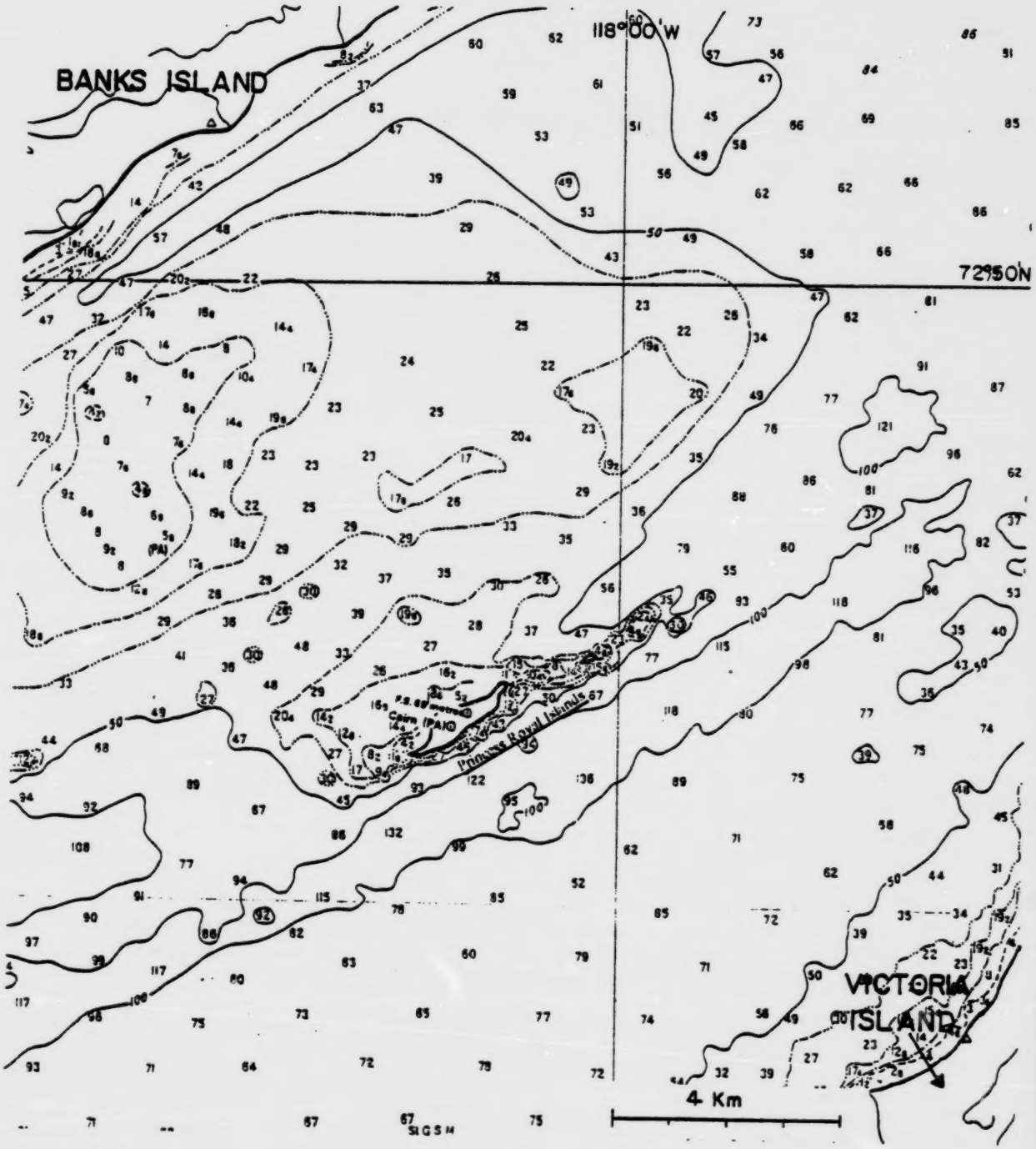

Figure 15: Princess Royal Islands Portion of Chart No. 7833, 1976 edition. Original scale 1:100,000; Depths in metres.

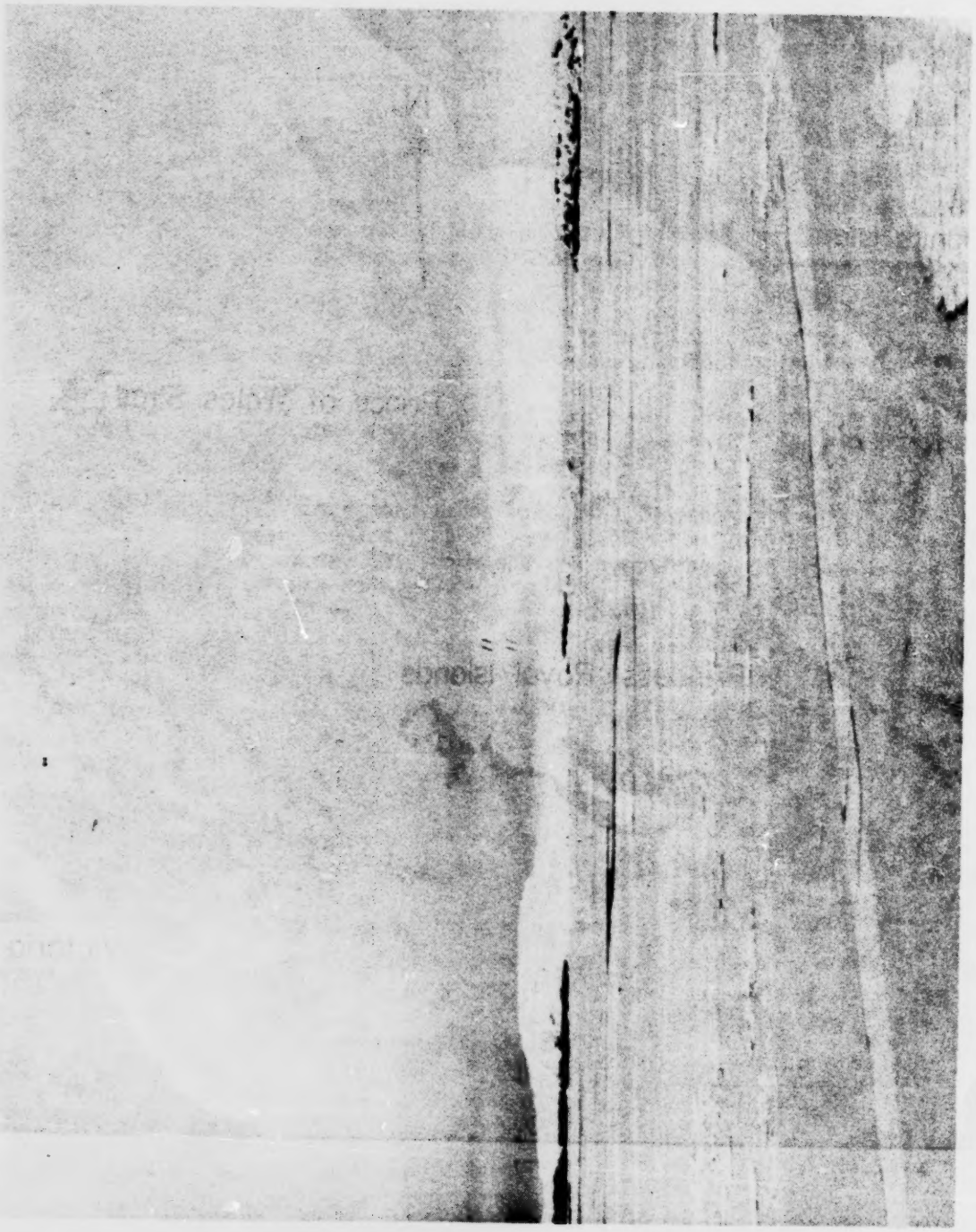

Figure 16: Princess Royal Islands Photograph taken by A. Allan, September, 1979.

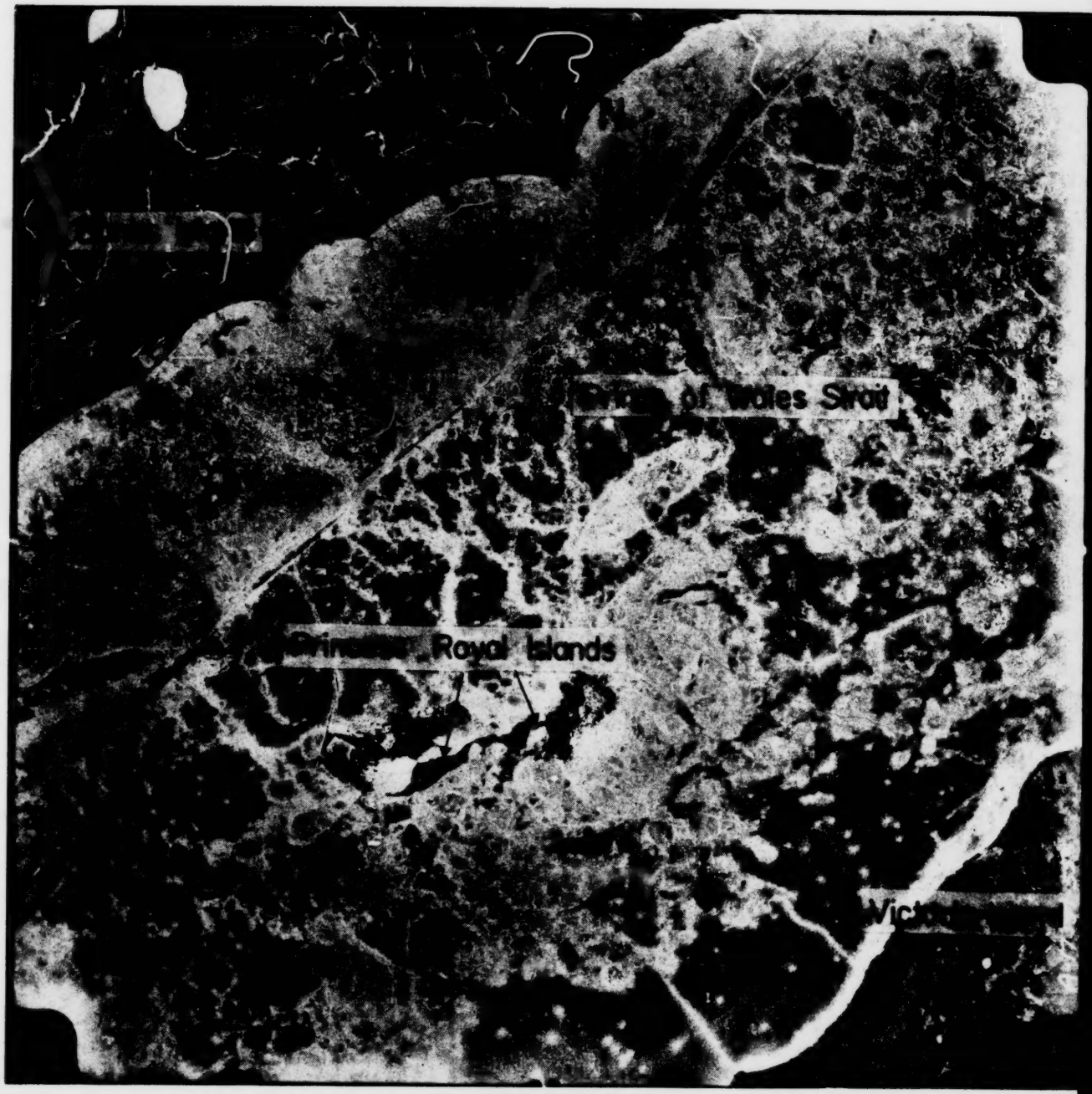

Figure 17: Princess Royal Islands Aerial photograph taken from an altitude of 9144 m with a 88 mm lens. June, 1961. Original photograph supplied by the Surveys and Mapping Branch, Department of Energy, Mines and Resources.

Table 9

Unnamed Islands off Ballast Beach, Banks Island — Latitude 74°29'N, Longitude 122°55'W

Chart No. 7832 shown in Figure 3

General Description: Two small islands that lie approximately 1.8 km off the northern coast of Banks Island about 59 km ENE of Cape Prince Alfred.

Water Depths: Deep water is found all along the northern coast of Banks Island. The 200 m isobath lies within 2 km of the coast. Soundings of 364 m and 397 m are charted to the NW and NE of the islands.

Surface Currents: No data available. A general eastward flow predominates through M'Clure Strait.

Ice Conditions: Heavy ice conditions with high concentrations of multi-year ice are typical year-round in M'Clure Strait. Shore leads may develop along the north coast of Banks Island in June and July.

Size and Shape: No data available.

2.6 The Eastern Arctic

The summaries of MYI concentrations in May and September shown in Figures 1 and 2 respectively indicate that much of this area does not meet the concentrations outlined in the selection criteria.

The Nares Strait extending from the head of Baffin Bay to the Lincoln Sea is one of the routes by which severely deformed ice from the Arctic Ocean drifts southward. Consequently, higher proportions of MYI are found here than in the remainder of the Eastern Arctic. Although frozen from shore to shore during winter, once breakup begins in mid-August the ice cover is variable and mobile until mid-September. The exposed capes and headlands throughout the Strait are subject to considerable ice pressure from a combination of strong southerly currents and the momentum of large floes driven by wind from the Lincoln Sea.

With the exception of spot soundings through the ice in Robeson Channel and a controlled survey in Smith Sound, most of the inshore waters have not been sounded. Currents predominately flow southwestward. Speeds up to 2 kts (1 m/sec) have been recorded through Robeson Channel (Fenco and Slaney, 1979). A net southward drift of ice of 16 km in 24 hours has been noted in Kennedy Channel (Fenco and Slaney, 1979).

Apart from Robeson Channel, much of the west coast of Nares Strait provides a number of islands and rocks near the headlands of deep indented bays and fiords. A number of these have been selected for inclusion in the report and appear to warrant closer examination. The geographic location of each of these areas is listed in Tables 10 and 11. No estimate of size or shape other than from the charts included in the report were available.

Table 10

Breakwater Island Latitude 81°42'N Longitude 64°50'W

Western Hall Basin, Chart No. 7072 shown in Figure 18

A small islet located off the eastern tip of Bellot Island in Lady Franklin Bay. This Bay is rarely completely clear of ice. North and Northwesterly winds frequently fill the Bay with heavy pack ice.

Skraeling Islands Latitude 70°55'N Longitude 75°41'W

Western Kane Basin, Chart No. 7071 shown in Figure 18

A number of small islets and rocks are located east of Skraeling Island at the entrance of Buchanan Bay that could be of interest to this study. There are no indications of water depths charted around these islets, however extremely deep water (over 100 m) occurs east of them. Shoal areas and foul ground are charted to the west and south of Skraeling Island so it is possible that this continues out to the islets being considered.

Cocked Hat Island	Latitude 78°49'N	Longitude 74°35'W
Payer Harbour	Latitude 78°43'N	Longitude 74°12'W
Neptune Rock	Latitude 78°38'N	Longitude 74°35'W
Cape Herschel	Latitude 78°35'N	Longitude 74°39'W

Northern Smith Sound, Chart No. 7071 shown in Figure 18

All of the above features lie off the east coast of the Johan Peninsula on Ellesmere Island in an area just north of the northern boundary of the North Water Polynya.

Cocked Hat Island is the largest island, just under 1 km north-south. The Pilot reports that it rises from a flat base to a conical summit with an elevation of 152 m.

Two of the unnamed islands that are situated at the entrance to Payer Harbour are less than .5 km in size and might be suitable platforms. The harbour is deep close to shore (greater than 37 m) and is reported to afford no shelter from winds or ice.

Neptune Rock is reported in the Pilot as low island about 4 km north of Cape Herschel.

Off Cape Herschel two submerged rocks have been reported. One about 1.8 km off the Cape was stuck by the Neptune in 1904 in an area of deep water (128 m). The second rock has not been positively identified but is reported to lie 11 km NE of the Cape off the entrance to Rosse Bay.

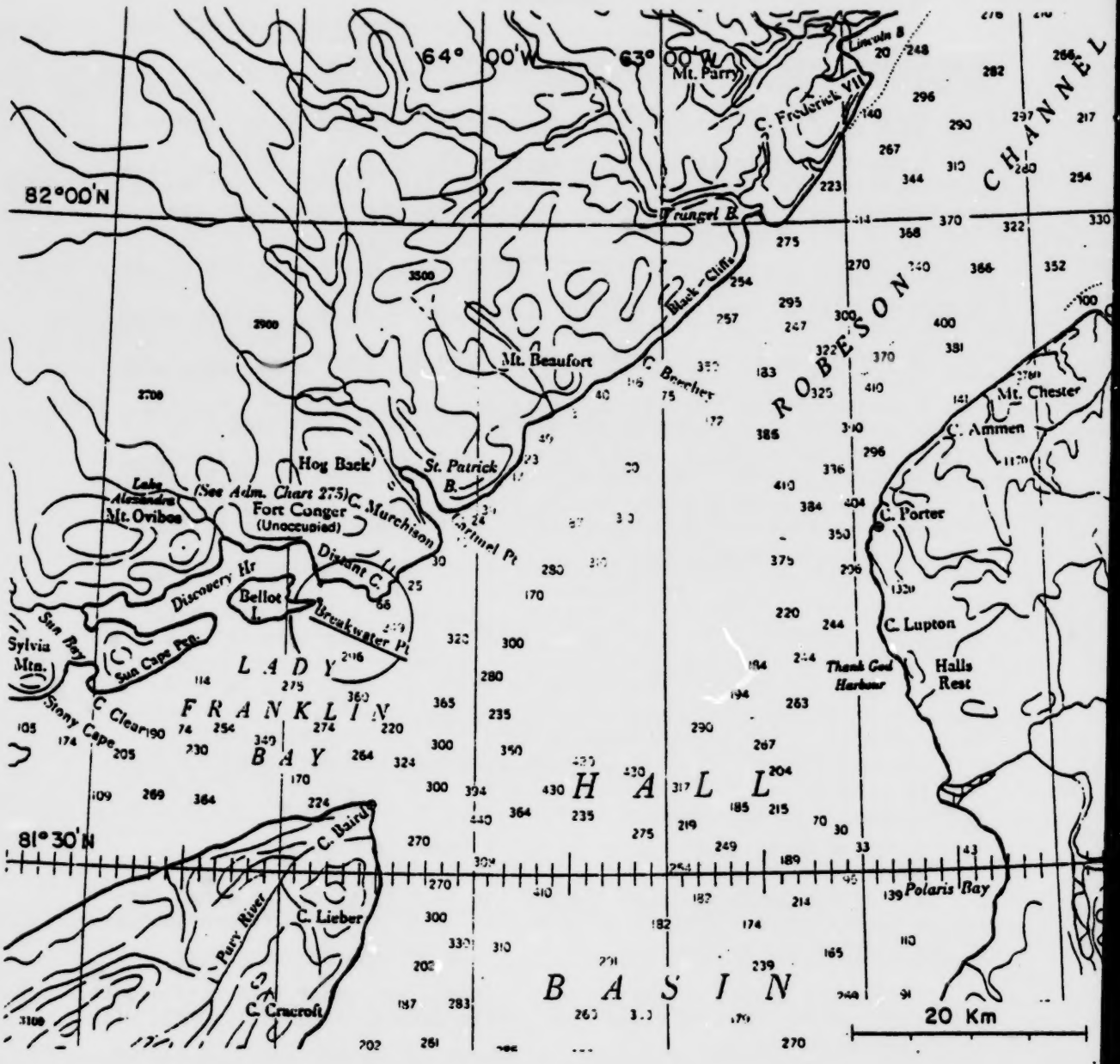

Figure 18: Breakwater Island Portion of Chart No. 7072, 1972 edition. Original scale 1:500,000; Depths in fathoms.

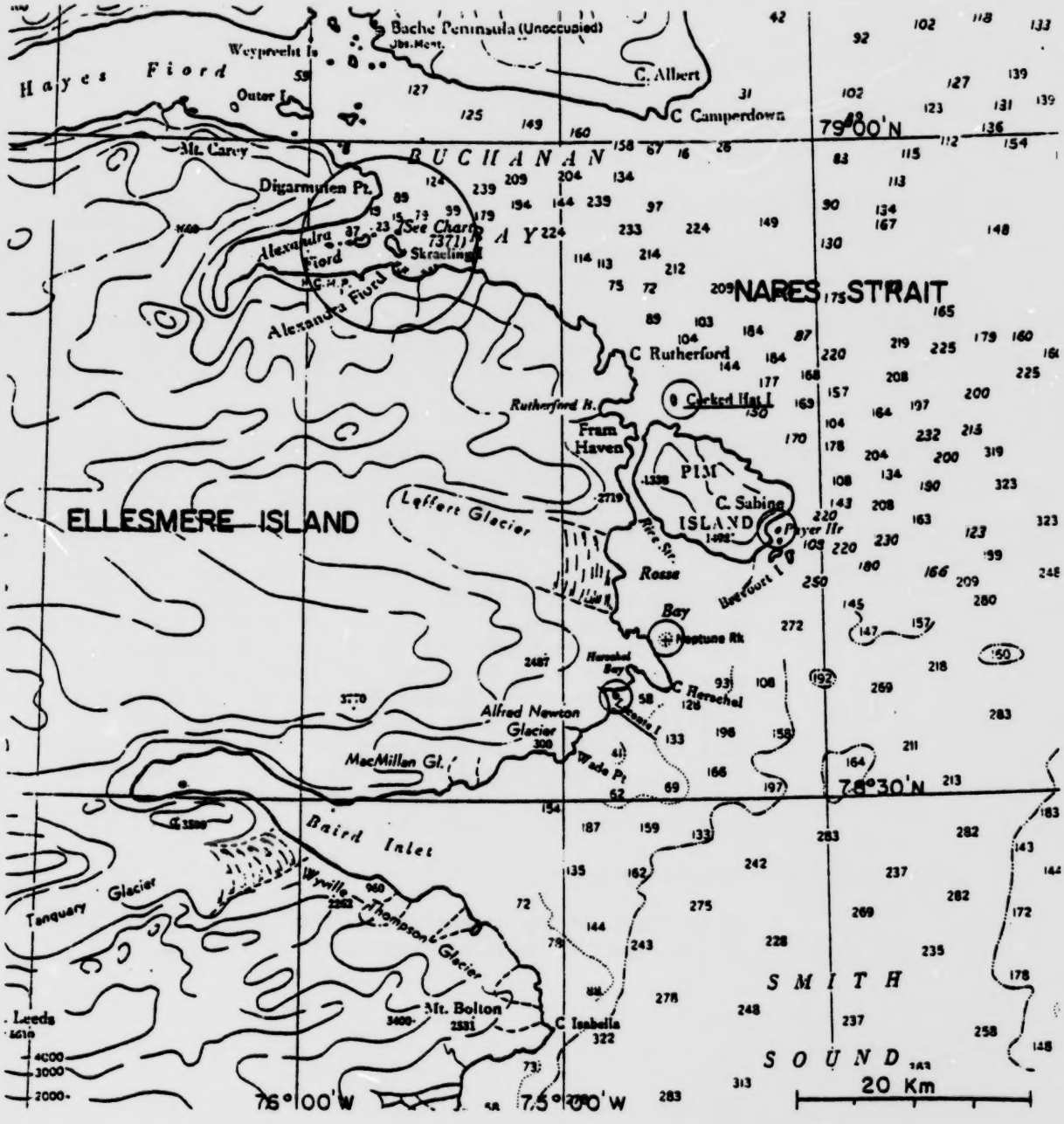

Figure 19: Skraeling Island, Cocked Hat Island, Payer Harbour, Neptune Rock and Cape Herschel. Portion of Chart No. 7071. 1972 edition. Original scale 1:500,000; Depths in fathoms.

Table 11

Carey Islands Latitude 76°37'N Longitude 73°00'W

Chart No. 7071 shown in Figure 19

General Description: A group of Danish island located WNW of Saunders Island off the west coast of Greenland. The group includes six islands and numerous islets and rocks. The latter being of interest as potential test sites.

Water Depths: There are no detailed soundings available in the immediate vicinity. Sudden changes of depth with shelving shoal rock that extends from one island to another are reported in the Pilot.

Surface Currents: East of the Carey Islands the currents flow northward while a stronger southward current lies to the west of them. A cyclonic circulation around the Careys has been measured (Meunch et al, 1969).

Ice Conditions: The Carey Islands lie within the semi-permanent polynya referred to as the "North Water". As such the winter regime is composed mainly of thin first-year ice rather than thick second-year and first-year ice that would be expected here. The Pilot for the area states that the Careys are rarely completely frozen over. A maximum of 7/10 to 9/10 coverage is indicated during the winter. Once the northern ice bridge at the head of Smith Sound breaks up, usually in July or August, the Careys may be exposed to the Southward drifting older ice from the northern reaches of Nares Strait.

Size and Shape: See photograph in Figure 21

No estimate of the dimensions of the islets or the depth of the reported rocks was available.

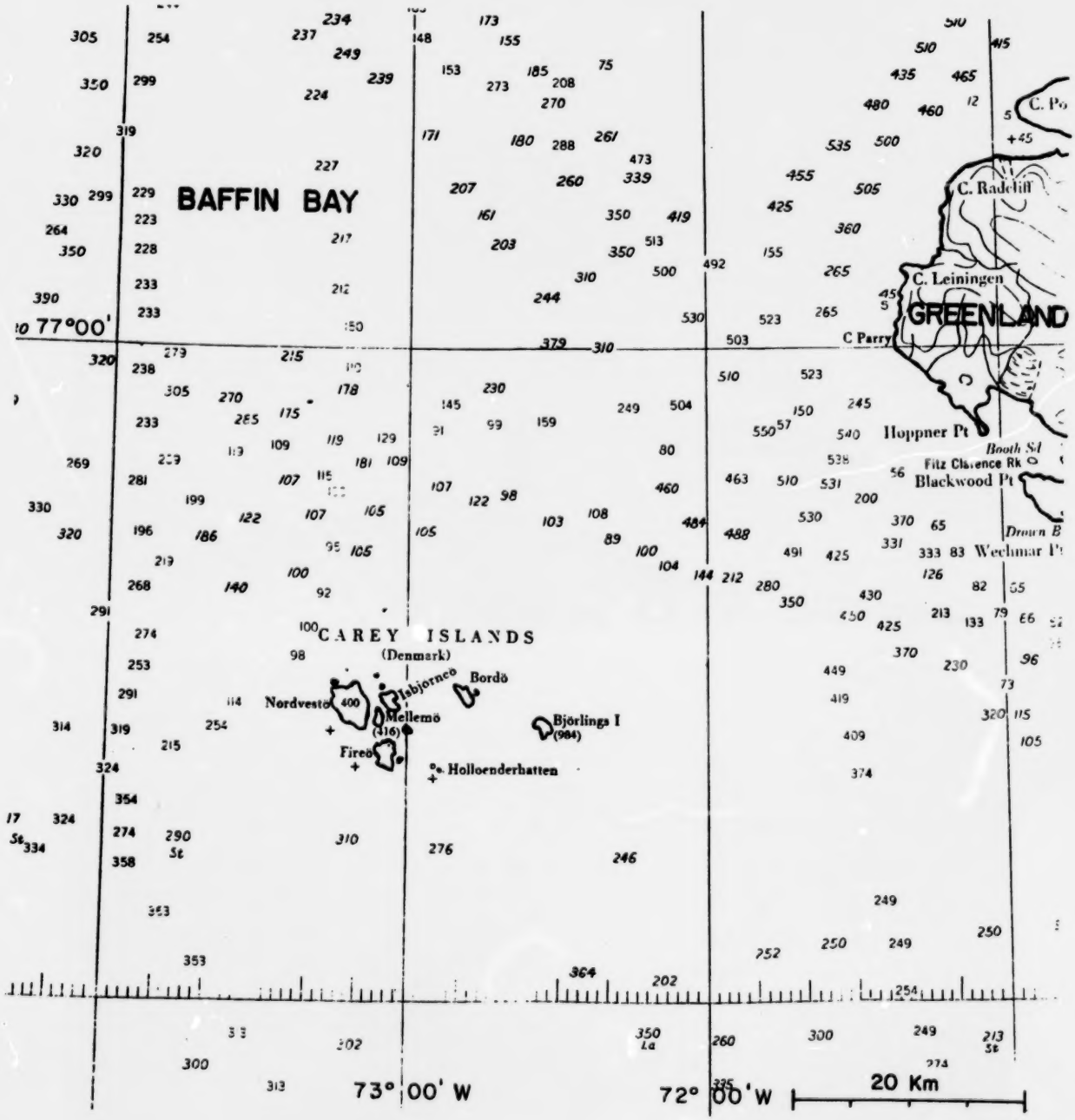

Figure 20: Carey Islands Portion of Chart No. 7071, 1972 edition.
Original scale 1:500,000, Depths in metres.

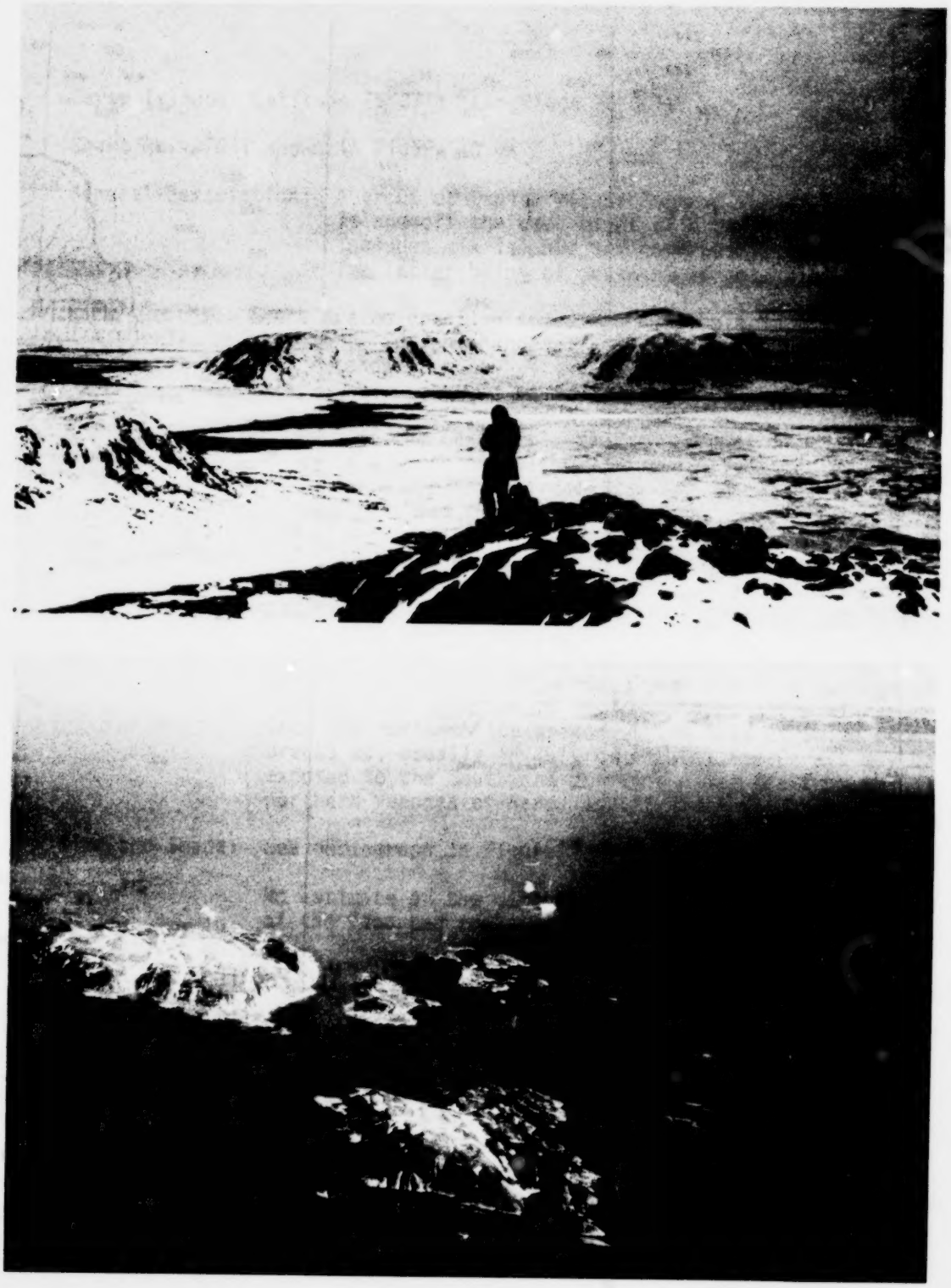

Figure 21: Carey Islands Upper photograph taken by K. Hay, May, 1973. Lower photograph taken by the Danish Government.

3.0 Conclusions

Given the number of fundamental characteristics missing from the site descriptions a critical evaluation of individual rocks or islands cannot be made. Some of the inputs required for evaluation, such as the sheerness of the island's shorelines could possibly be ascertained from a single low altitude overflight. It would be expected that the choice of platforms would then be narrowed considerably so that in-situ measurements of water depths and currents would become more practical.

It is recommended that priority be given to the sites located in the Beaufort Sea or Arctic Ocean. The islets and submerged rocks identified in this area would provide a test platform exposed to ice forces very close to the worst case that operational drilling platforms will face.

The sites identified in the Archipelago and the Eastern Arctic do not offer any substantial advantages in terms of logistics nor do they as closely approximate the conditions to be tested.

A test program operating from any of the sites identified will involve considerable logistics problems. In view of this, a site in Eastern Canada, possibly along the coasts of Labrador or Baffin Bay, should be considered for preliminary programs to evaluate equipment and testing techniques. Among the advantages offered by a test site in the lower latitudes are ease of access and fewer problems in transfer of equipment and personnel; proximity to major centres or supply bases; more favourable working conditions in terms of the number of daylight hours and weather conditions.

References

Fenco Consultants Ltd. and F.F. Slaney and Company Ltd., 1978, An Arctic Atlas: Background Information for Developing Marine Oilspill Countermeasures, Arctic Marine Oilspill Program Report. EPS-9-EC-78-1 Environmental Impact Control Directorate Fisheries and Environment Canada, August 1978.

Gov't. of Canada, 1968, Pilot of Arctic Canada, Vol. III, Second Edition, Canadian Hydrographic Service, Dept. of Energy, Mines and Resources, Ottawa.

Gov't. of Canada, 1970, Pilot of Arctic Canada, Vol. I, Second Edition, Canadian Hydrographic Service, Dept. of Energy, Mines and Resources, Ottawa.

Gov't. of Canada, 1978, Sailing Directions, Arctic Canada, Vol. II, Third Edition, Dept. of Fisheries and Environment, Ottawa.

Meunch, R.D. et al., 1969, The Baffin Bay - North Water Project, Report No. 4, AINA Research Paper No. 52, March 1969.

Oceanographic Services, 1969, Environmental Analysis of the Canadian Arctic Archipelago Part 2 - Oceanography and Hydrography. Contract Report prepared for Global Marine; Santa Barbara, Calif.

Weeks, W.F., 1976, Sea Ice Conditions in the Arctic, in Aidjex Bulletin No. 34, Dec. 1976, pp. 173-206.

FEASIBILITY STUDY FOR A COLD OCEAN TEST STRUCTURE (COTS)

by

D.V. Reddy and A.K. Haldar

Faculty of Engineering and Applied Science

Memorial University of Newfoundland

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Nomenclature

F_D = drag force per unit length

U_h = air velocity in the undistributed flow at ridge height, h

C_{Dh} = form drag coefficient

R_D = mean ridging density

ρ_a = air density

p(h) = Probability density function of the ridge height

h_o = lower cutoff height below which surface friction plays predominant part.

F_w = wind force

C and C_s = shape coefficient

α = angle between wind direction and the surface of the member

V_n = wind speed or current speed

A = projected area of the member

V_z = total current speed at a distance z above the bottom

ζ = density of water

F_e = current force

C_D = drag coefficient

F_I = Inertia force per unit length.

$U \varepsilon \dot{U}$ = velocity and acceleration of water particle

D = diameter of the pile

C_D and C_M = drag and inertia coefficient

V = ice velocity

$\dot{x}$ = structure velocity

$\dot{\sigma}$ = stress rate

σ_c = crushing strength of ice

θ = polar angle of the pile when the ice crushes

R_H = Horizontal force on the cone

R_V = Vertical force on the cone.

γ_w = weight density of water

σ_f = flexural strength of ice sheet

t = ice sheet thickness

u = coefficient of friction between ice and the structure.

ζ = resolution factor

w = orbital width

FEASIBILITY STUDY FOR A
COLD OCEAN TEST STRUCTURE (COTS)

1.0 OBJECT

The purpose of this project is to propose a Cold Ocean Test Structure (COTS) for ice force measurements in the Beaufort Sea. Based on relative displacements of the ice and structure, the concept will be evaluated for both framed and gravity type structures. Particular references will be made to the determination of the need for a fairly rigid structure. This proposal will be divided into the following parts:

i) review of literature,
   a) environmental forces
   b) existing test platforms
ii) concept evaluation,
iii) proposed design and cost estimate,

and iv) conclusions.

2.0 INTRODUCTION

The need for realistic values of ice pressures has been indicated by several investigators. Opinion is divided on the 'conservatism' associated with the values used in current design practice. In view of the parasitical tendency of a heavier structure to increase the loading and the structural costs, there is a noticeable trend now to consider structures more slender than the monopods. The slender con-

cept introduces the problem of the effect of the period of the structure on the response, dynamic magnification factors, self excited oscillations, etc. Thus any slender test facility should provide for the measurement of the dynamic effects of ice loading. However, in view of the very large forces associated with ridge loading, the slender structure is an unacceptable concept. Even the more rigid framed structure is not suitable for a test platform in an ice environment. The following observation made by Liesemer (1980) is very relevant:

"Although the fixed platform concept has not yet been ruled out, it is believed that, based on reservoir geometry and reservoir permeability, a production scheme involving massive fixed platforms cannot be economically justified. The alternative is to utilize floating production facilities which can be rapidly disengaged and moved out of the area when threatened by icebergs or massive ice floes".

The first production platforms in ice-infested waters were those installed in Cook Inlet, Alaska in the early 1960's. The typical structure was a fixed based, framed type in 20 m water depth. A 9 m diameter monopod has also been built. The design ice pressures were of the order of 2.09 MPa. Some of the Cook Inlet structures were instrumented to measure ice forces particularly with reference to dynamic effects. Other available test data are from lighthouses in the St. Lawrence and the Gulf of Bothnia, Finland.

This report will compare alternate concepts, outline the basic platform design, and describe the techniques used to estimate the design loads. Also, the procedures for measuring the strains and ice forces

will be described.

The proposed cold ocean test structure (COTS) will be a large scale experimental facility for evaluating ice forces on an applicable fixed base structure, i.e. the monopod. This is further discussed in a later section of this report. The data obtained will include static and dynamic ice forces experienced by the structure.

3.0 REVIEW OF LITERATURE

3.1 Environmental Forces

The environmental phenomena to be considered in addition to forces due to moving multi-year ice ridges are forces due to winds, waves, currents, tidal fluctuations, earthquakes, tsunamis, tornadoes, soil conditions, air and water temperatures, fatigue and corrosion. During the assessment of these factors on an ocean structure, the motion of the structure must be taken into account.

3.1.1 Ice Loading

3.1.1.1 Ridges:

The most important factor to be considered for ice loading in the Beaufort Sea is the multi-year ridge. Ridges, which are caused by the compressional and shearing interactions of ice floes, extend several metres above and below the ocean surface. Reports vary as to the amount of ice cover occupied by ridging. Fig. 1 gives a schematic of typical winter ice conditions in the Beaufort Sea. Estimates of 22% - 39% in the Kara Sea, and 18% in the Canadian basin have been reported by

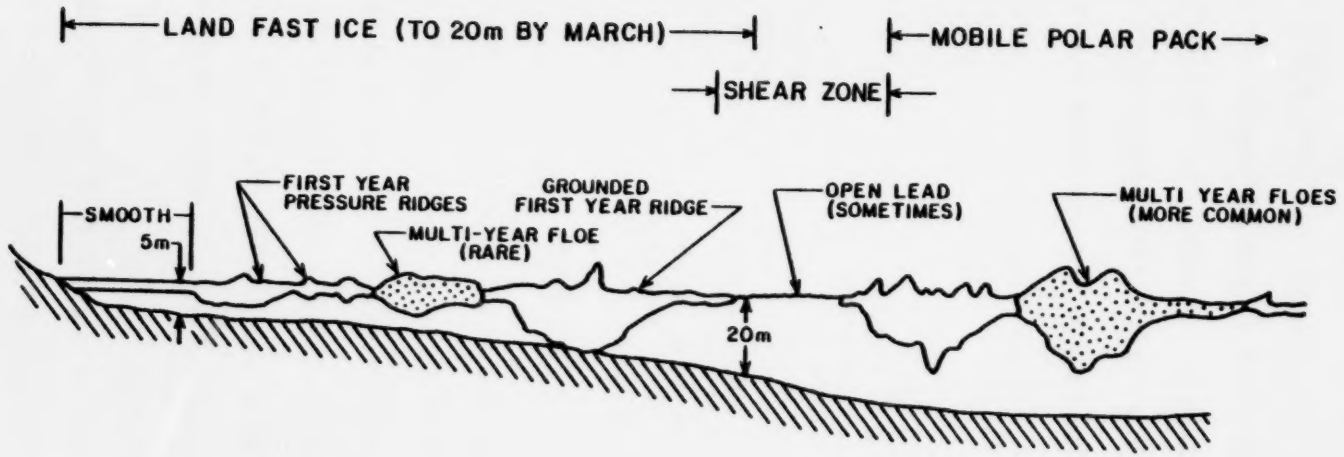


Figure 1 Typical Winter Ice Conditions, Beaufort Sea (Croasdale, 1977).

Kirillov (1957), and Wittman and Schule (1966).

Parmerter and Coon (1972) used a computer programme to simulate ridge formations in ice thicknesses from 0.2 m to 2 m. The computed ridge profiles indicated thicknesses of about 22 m. Banke, Smith and Anderson (1976) have pointed out that wind drag on ridges may be viewed as form drag and parameterized in terms of the sail area in a vertical plane. Arya (1973) derived an expression for the form drag per unit area in terms of certain key parameters of ridge statistics and a suitable drag coefficient. As wind-induced stress is the principal activator of ice movement, ice forces due to large ridges can be of a dynamic nature. In a state-of-the-art paper on research and development of ice engineering, Croasdale (1977) suggested typical structures viable for the Beaufort Sea condition as mobile gravity - founded monopods and conical structures. The complexity of designing an offshore drilling platform to withstand the loading from moving multi-year ridges was pointed out and empirical formulae presented to calculate these loads. Ridge loading was indicated to be about 10 to 15 times that due to the parent ice sheet. The most practical calculation of forces due to ridges are those of Ralston (1977) which are described later as part of the analysis of the proposed concept.

3.1.1.2 Ice Sheets:

For forces due to ice sheets, which are considerably less than those due to the ridges, reference must be made to the detailed review papers of Korzhavin (1962), Neill (1976), Danys (1972), and Reddy, Cheema, and Swamidas (1975). The work related to the generation of artificial force records, described by Swamidas, Reddy, and Purcell (1976) can be adapted for ridge loading.

3.1.1.3 Design Loading:

Gerwick's (1970) excellent investigation of the effect of the structure geometry on the forces induced by ridges and ice sheets can be summarised as follows:

1) Vertical Face

a) Ice Sheets

The typical ice failure on a vertical wall is of a crushing nature. A relatively long wall with an aspect ratio (b/h = 15), or more, would be subjected to loads of 2.153 MPa for an ice sheet 3.05 m thick whereas for a slender vertical wall of say 10.67 m in width, with b/h ratio of 4, the same ice thickness would cause a loading of 4.545 MPa. Both values are based on a dynamic load factor of 1.5. The experience at Cook Inlet, Alaska, indicated the need for considering the vibration characteristics of the ice failure assuming an oscillation period ranging from 0 to 1.5 seconds.

b)Pressure Ridges

Typical loads from a pressure ridge, Fig. 2, on a slender, vertical wall type structure, Fig. 3, would be higher than those from a parent ice sheet. The conventional procedure is to multiply the load calculated from the ice sheet on the structure by a ridge factor. Cook Inlet experience suggests a ridge factor of 1.5 to 3.6 times the loads recorded from the parent sheet. The typical ridge load would, therefore, be based on the previous parent ice sheet load calculation, 10.048 MPa.

2) Cylindrical Structure

a) Ice Sheets

Loads due to an ice sheet impinging on a vertical cylindrical struc-

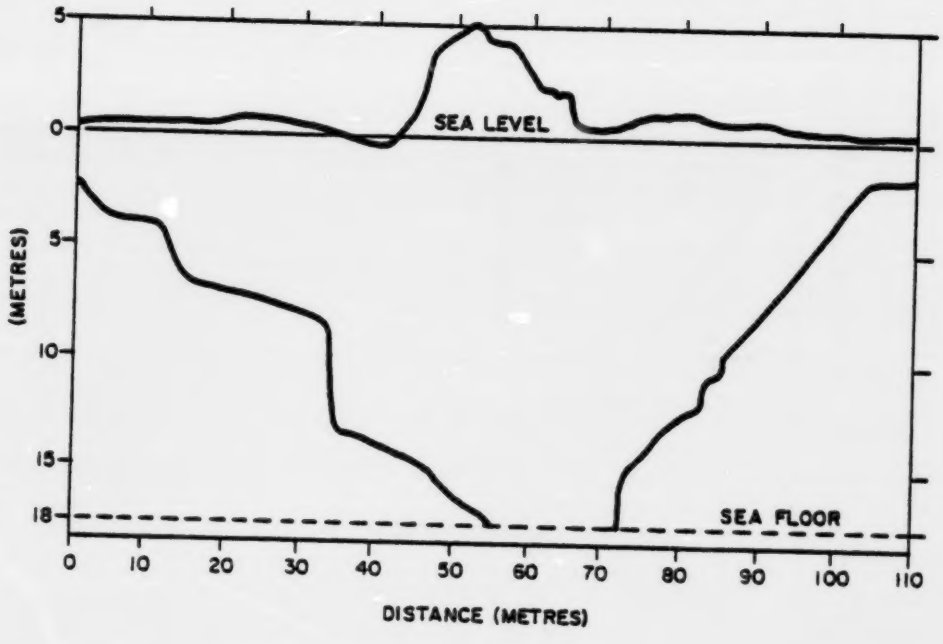


Figure 2 Profile of Arctic First Year Ridge (Croasdale, 1977).

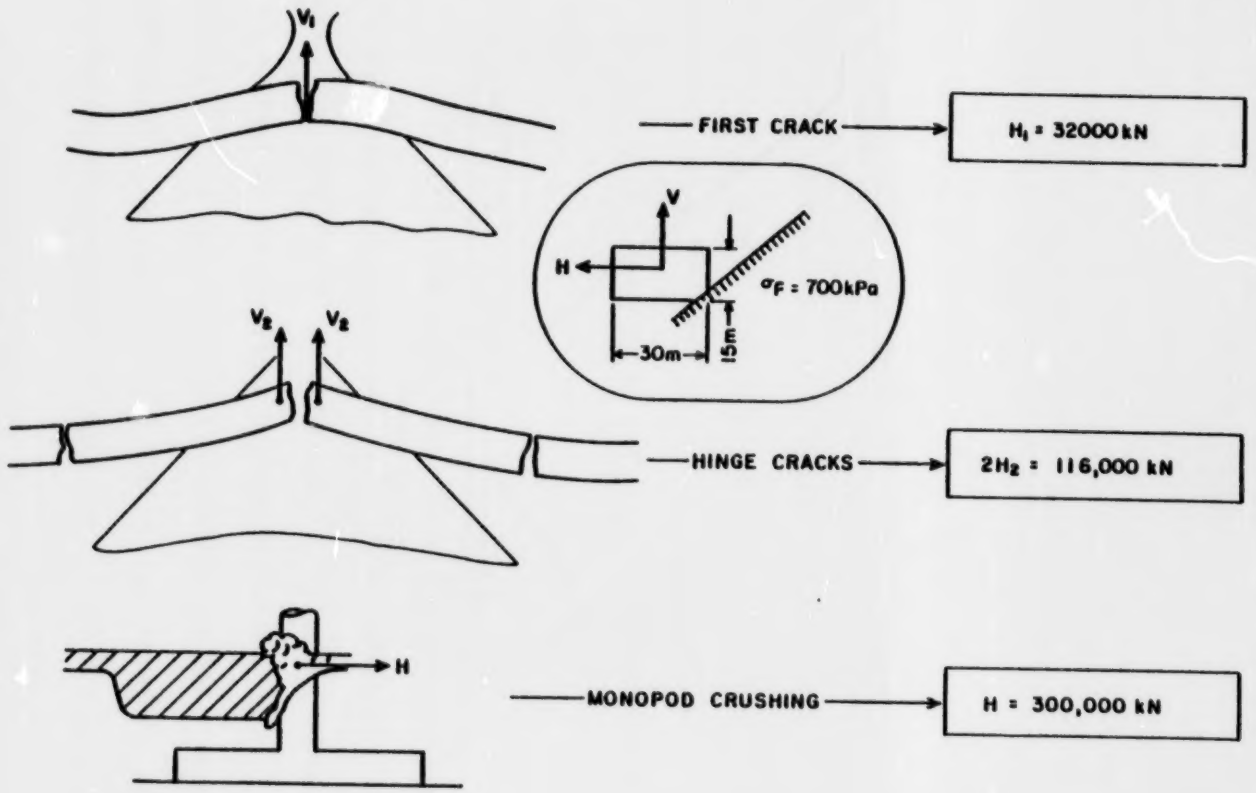


Figure 3 Typical Ridge Loads (Croasdale, 1977)

ture will be 10% less than that of a flat surface because of the influence of the shape.

b) Pressure Ridges

The 10% reduction will still apply and the ridge factor for this case will be 2.2.

3) Conical structure

a) Ice Sheets

Failure of ice sheets on a conical structure will be of the bending type predominantly; the shape will cause ice ride up for a cone with the apex above.

b) Pressure Ridges

A typical force value exerted by ridges on a 9.15 m diameter, vertical cylindrical structure will be 146,666 kN; for a 4.57 m diameter similar structure the force will be of 93,333 kN. For a 22.87 m cone diameter at water level changing into a 9.15 m diameter cylindrical shaft, the horizontal load will be 48,888 kN for the most severe jam up with a realistic value of 35,555 kN.

3.1.2 Wind Loading

The forces due to wind will be considerably less than those caused by ice sheets and ridges; for fixed offshore structures, wind may contribute up to 5% of the total loading. However, wind can have a significant effect on the forces exerted by pressure ridges. This section will, therefore, be primarily concerned with effect of wind on ridges, and wind loading on the tower will be discussed only briefly.

3.1.2.1 Wind Forces on Ridges:

The transfer of momentum from the atmosphere to the ice surface consists of two parts: the skin friction drag on the level surface and form drag on hummocks and pressure ridges. Hibler, Weeks, and Mock (1972) expressed the total form drag per unit area, assuming ridging to be homogeneous and isotropic for a given area, in the following form:

$$F_D = \int_{h_o}^{\infty} C_{Dh}\, \rho_a\, U_h^2\, h\, P(h)\, dh \quad \frac{R_D}{\pi} \qquad \ldots(1)$$

where

C_{Dh} = form drag coefficient = $\dfrac{f_D}{h(\frac{1}{2}\rho\, U_h^2)}$,

in which

F_D = drag force per unit length of a ridge oriented normal to the flow,

U_h = air velocity in the undistributed flow at ridge height, h,

R_D = mean ridging density.

ρ_a = air density,

$P(h)$ = probability density function of the ridge height,

and h_o = lower cut-off height below which surface friction plays the predominant part.

The most uncertain parameter is the form drag coefficient, C_{Dh}, for which a value of 0.4 has been assigned.

3.1.2.2 Wind Forces on the Structure:

The effect of the wind forces on the structure are additive to those caused by ridge and ice forces. Two kinds of wind speeds, namely

sustained and gust wind speeds, have to be considered to take care of the static and dynamic effects. These wind speeds are given by offshore codes of practice such as Det Norske Veritas (1974, 1975).

Once the wind speed is determined, the wind force normal to flat surfaces or normal components to the axis of members not having flat surfaces, can be calculated using the following formula:

$$F_W = CA \frac{V_h}{16} \sin \alpha \qquad \ldots(2)$$

where

F_W = wind force in Kgf,

C = shape coefficient,

α = angle between wind direction and the surface of the member,

V_h = wind speed in ms^{-1}

and

A = projected area of the member in sq. metres - taken as the normal projection on a plane normal to the direction of the force.

The alternative relationship for the calculation of wind forces on concrete ocean structures has been given by the Federation International de la Precontrainte (1974) as

$$F_W = 0.0625\ C_S\ AV_h^2 \qquad \ldots(3)$$

where the value of C_S for beams is 1.5, for the sides of the building, 1.5, and for cylindrical sections, 1.0.

3.1.3 Current Loading

For the environment of the Beaufort Sea, currents can have significant effects on foundation design. Current forces cause sizable

drag loads, siltation and erosion of the seabed, and may cause fluctuating loads due to vortex shedding and induce oscillations reducing the fatigue life drastically. The variation of current speed with depth can be determined from the following relation (Det Norske Veritas 1974, 1975):

$$V_Z = (V_h)_1(Z/h)^{1/7} + (V_h)_2(Z/h) \qquad ...(4)$$

where

V_Z = total current velocity at a distance Z above the bottom,

$(V_h)_1$ = resulting velocity, at still water level, of currents not generated at the surface (e.g. tidal currents),

$(V_h)_2$ = resulting velocity, at the still water level, of currents generated at the surface (e.g. wind generated current),

and

h = water depth, measured from the still water surface.

In open areas, the wind generated current speed can be taken as

$$(V_h)_2 = .01\ V_{10} \qquad ...(4a)$$

where V_{10} is the sustained wind speed at 10 m above the still water level. The tidal current speed $(V_h)_1$ varies from 0.5 m/sec to 1.5 m/sec depending on the latitude of the site. Once the current speed is determined, the current forces can be calculated using a similar approach to that for wave force calculations. The drag and lift forces due to currents can be determined using the formula:

$$F_e = 1/2\ \rho_w\ V_z^2\ C_D\ A, \qquad ...(5)$$

where

V_Z = design current speed,

ρ_w = density of water,

C_D = drag coefficient determined from model tests,

and

A = projected area of the member.

In cases, where superposition of current and wave effects is possible, the current velocity should be added vectorially to the horizontal wave particle velocity before the drag force is calculated. Brunn, Houbm, Rye and Saetre (1976) considered the collection and analysis of wind, wave and current data, and the need for further collection of such data more important than the development of new analytical techniques. Tidal fluctuations of sea level are of considerable importance in the design of offshore structures, especially when the inclusion of surge is necessary. Prediction of surges in offshore areas cannot be effected in isolation; a statistical correlation of tide, storm surge and current is required.

3.1.4 Other Loading

As indicated by Stenning and Schumann (1979), structures built in the Beaufort Sea will be subjected in summer months to wave loading. Also, the sea site is in seismic zone 3 according to the National Building Code of Canada. Wave and seismic loads are dynamic in nature, and may cause a considerable amount of amplification of the response if the structural frequency is very close to the forcing frequency. Therefore, dynamic analyses need to be carried out for proper evaluation of the response quantities. But the responses due to the ridges could be treated in a static manner; this

will be described in the later part of this report. The following section will describe the wave loading calculation procedures similar to those of ice, wind or current as described previously.

3.1.4.1 Wave Loading reproduced from Babu and Reddy (1977):

Wave conditions may be described either by means of wave energy spectra or deterministic design waves having appropriate shape and size. The wave forces acting on the ocean structures can be divided into the following components as described by Hooft (1971): a) Froude-Krylov forces due to the pressure distribution over the structural part, in a wave that is not disturbed by that part, b) inertia forces due to the acceleration of the added mass of the structural part, in a wave that is not disturbed by that part, and c) damping forces due to the damping of the velocity of the water particles in the wave by the structural part.

Extreme waves, or 'design waves', are currently predicted by extrapolation of typically, one year wave data to provide a 'design wave', the height of which will be exceeded, on an average, once in 'N' years, where N may be 50 or 100. The size and shape of the structure and water depth affect the estimation of design wave height. A design wave based on extreme conditions and having a period of over 10 sec. does not necessarily cover all possible situations. With increasing water depths the structural frequencies will be reduced. For steel-frame structures in 30 m water depth, the frequency is typically 1.0 Hz, whereas in 120 m water depth this is decreased to 0.3 - 0.2 Hz; these frequencies correspond to those at which there is a significant amount of wave energy under conditions

of short wind duration, short fetches, and relatively low wind velocities. In such situations, the shape of the energy spectrum of the sea surface is essential to any method of estimating the dynamic response of a fixed structure; very little information exists on spectral properties under these conditions of non-fully developed seas. The recent results of the JONSWAP programme (Hasselmann, 1973) provide some information in this respect but further work is necessary.

Wave spectra are also required for predicting the fatigue life of offshore structure. In a recently developed method of predicting long-term loads on fixed platform members, more reasonable than earlier procedures, the starting point of the computation is the long-term bivariate histogram of $H_{1/3}$ and T_Z presented by Draper (1970). Here $H_{1/3}$ is the significant wave height and T_Z is the average zero-up-crossing period (average apparent wave period). A particular pair, $H_{1/3}$ and T_Z, may then be used to compute the corresponding stress in the critical section of a platform; the stress is allotted the same probability of occurrence as the $H_{1/3}$ and T_Z pair from which it is derived. Different wave theories have been developed to determine the velocities and accelerations of the water particles in the wave, and the range of validity of these theories cannot be precisely described. Bretschneider (1969) has suggested one criterion for selecting a particular wave theory on the basis of the value HL^2/h^2, where H is the wave height, L is the wave length, and h is the water depth, which enables a choice between sinusoidal, conoidal, and solitary wave theories as shown in Fig. 4. In problems where the drag force is very important, higher order wave theory may be used.

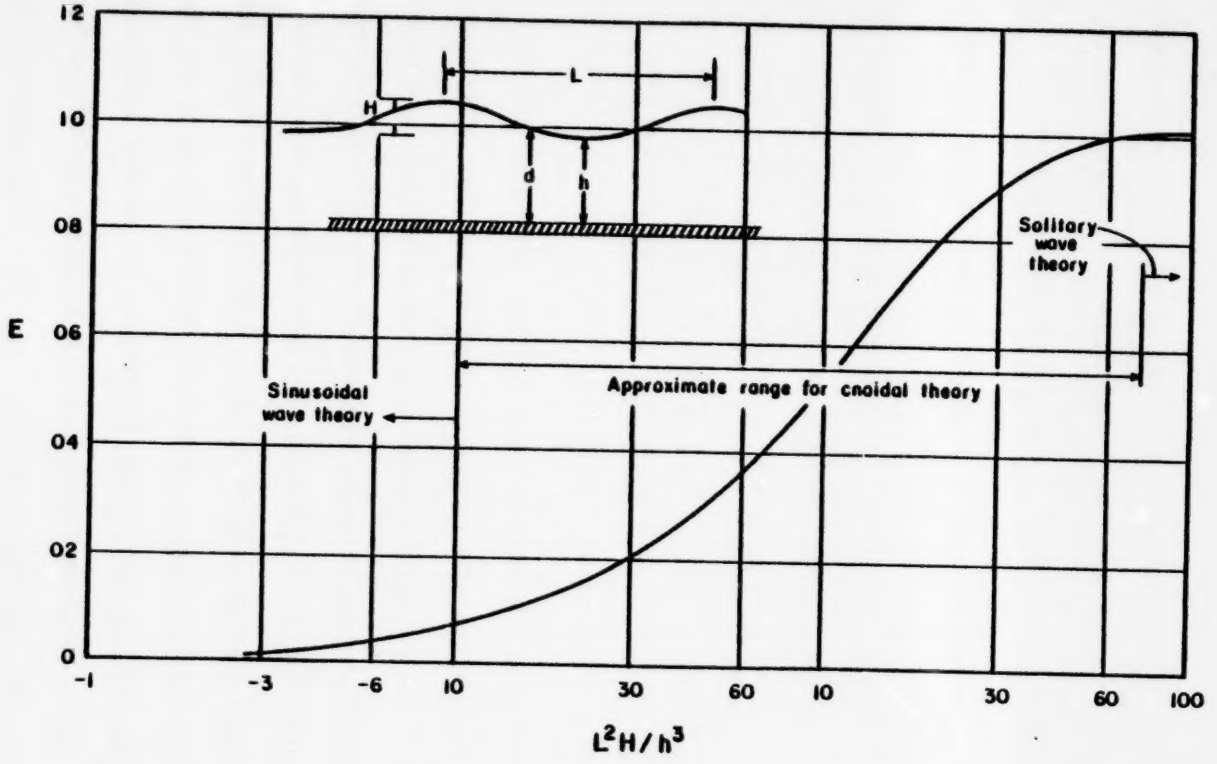


Figure 4 Variation of Elliptic Parameter m as a Function of L^2H/h^3 (Bretschneider, 1969).

Asgard and Dean (1969) provide guidelines to select the required order of wave theory for better evaluation of water particle velocities and accelerations. The choice depends on water depth, wave height, and wave period, as shown in Fig. 5.

The well-known Morison equation provides the basis for relating forces on members to wave-induced velocities and accelerations in terms of a drag force proportional to the square of the velocity, and an inertia force proportional to the acceleration, with the corresponding drag and inertia coefficients C_D and C_M. In mathematical form the equation is expressed as follows:

$$F = F_D + F_I = \tfrac{1}{2} \rho_w C_D D U|U| + \rho C_M \frac{\pi D^2}{4} \dot{U} \qquad \ldots(5a)$$

where

F_D = drag force,

F_I = inertia force,

ρ_w = water density,

U, $\dot{U}$ = velocity and acceleration of water particle,

D = diameter of the object,

and

C_D, C_M = drag and inertia coefficient.

The ratio of the drag force to inertia force is given by $(H/D)(C_D/\pi C_M)$ in deep water, where H is the wave height and D is the member diameter. An appropriate criterion for the neglect of drag force is $D/H > 0.2$. Forces resulting from diffraction effects (for large diameter structure) may be considered significant for $D/L > 0.2$, where L is the wave length. If one assumes the 'design wave' has a steepness, H/L, of approximately 9/16, the latter criterion becomes $D/H > 3.2$. For a design wave of 30 m,

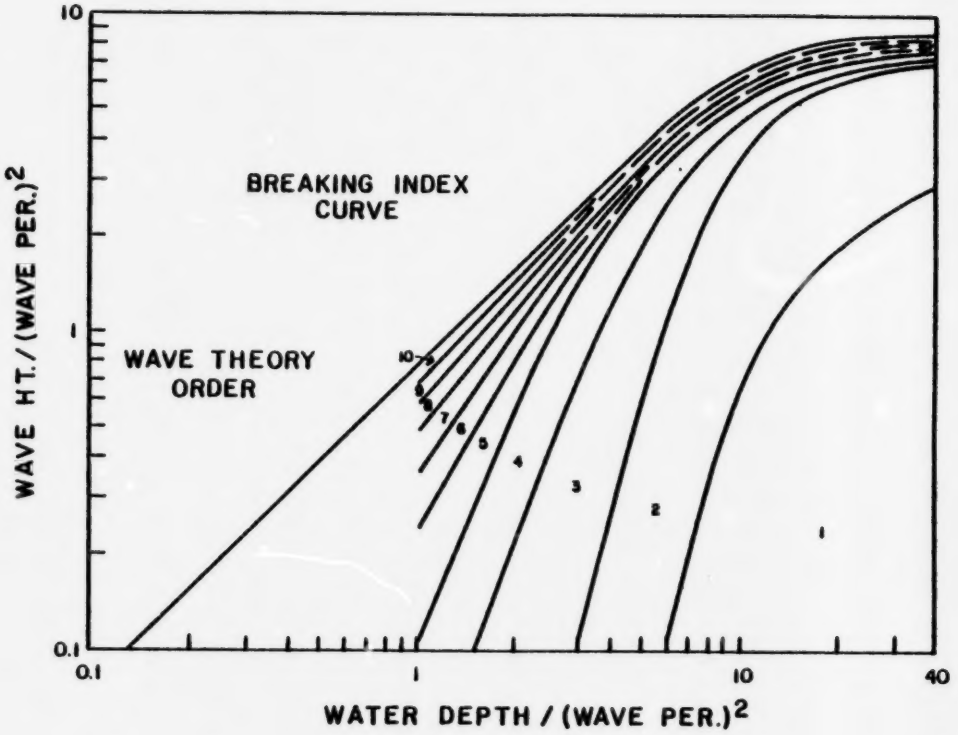


Figure 5 Recommended Order of Wave Theory (Asgard and Dean, 1969).

the drag force would be negligible for D > 6 m, and diffraction would be significant for D > 100 m. If H = 10 m, the corresponding values of diameter are D > 2 m and D > 32 m.

Hogben and Standing (1974) have recently developed computational procedures for large diameter (particularly gravity type) structures, based on the diffraction theory proposed by MacCamy and Fuchs (1954), which shows excellent agreement with laboratory tests. Bratteland, Leviseth and Torset (1972) calculated the wave forces components on a semi-submersible using the following relations:

$$F_1 = \zeta V \dot{U} \text{—Froude Krylov,} \qquad \ldots(6)$$

and

$$F_2 = C_M \zeta V(\dot{U} - \ddot{X}) \text{—Inertia,} \qquad \ldots(7)$$

$$F_3 = 0.5\, \zeta C_D A\, (U - \dot{X})\, |\, U - \dot{X}\, | \text{—Drag} \qquad \ldots(8)$$

where

V = submerged volume of the member,

A = projected area of the member,

ζ = water density,

C_M = added mass coefficient,

C_D = drag coefficient,

and

$U, \dot{U}$ = water particle speed and acceleration in the x-direction,

$\dot{X}, \ddot{X}$ = velocity and acceleration of the structure.

The total horizontal wave force in the x-direction is:

$$F_X = F_1 + F_2 + F_3. \qquad \ldots(9)$$

An extensive literature review concerning the estimation of drag and inertia coefficients to be used in the well known Morison's equation

for different offshore structures has been presented in a report by the British Ship Research Association (1976). Det Norske Veritas (1975) has presented the procedure for estimating the wave forces on large objects including semi-submersible floating platforms.

The conclusion to be drawn from this very brief summary of wave loading on offshore structures is that adequate methods are available for predicting such loads. The most important concern is the proper use of wave theories to compute the possible velocities and accelerations in waves. In deep water, Stoke's fifth order wave theory is used, although the resulting forces and moments differ only slightly from those derived from linear wave theory.

3.1.4.2 Earthquake and Tsunami Loading [reproduced from Babu and Reddy (1977)]:

Earthquake effects consist of two types of force-producing action which must be considered in the design of offshore structures. These actions are ground motions and impulsively generated waves or tsunamis. Earthquakes in offshore sites can cause ground shaking, landslides, liquefaction and soil loosening, faulting and ground rupture, tectonic uplifts or subsidence, and tsunamis.

Earthquake prediction is based on the proximity of the structure to major fault zones and historical records of earthquake occurrence and intensity. These records, when available, may be treated by extreme value theory to determine the maximum probable intensity to be expected within the design life of the structure which may be in the 50-100 year range. Ground motions caused by an earthquake transmit motions to a structure through its foundation. This accelerated movement from an at-

rest position causes the structure to be loaded by inertia forces which are a function of its flexibility, mass and damping characteristics (Hudson 1965). Alford, Housner and Martel (1964) presented spectra from 88 earthquakes, with the typical earthquake used for design being based on the one at El Centro in 1940 which had a maximum horizontal acceleration of 0.32 g.

Offshore framed platforms are made up of large diameter tubes interconnected by underwater bracing and fixed to the seabed by piling. As the submerged parts of a structure are forced to follow the earthquake motions, some water in the vicinity of the structure will move with the members. The mass to be used in the calculation of earthquake forces should include the mass of the structure plus that of any fluid it contains, and the added mass of water accelerated by the structure. A more difficult problem is the behaviour of foundation soils and piling during an earthquake. If the foundation material consists of saturated sand, liquefaction may occur during earthquake vibration as a result of the increased porewater pressure. Should this situation arise, two effects must be considered: (i) the resistance of the soil may decrease to nearly that of the fluid making the apparent point of support move lower as the oscillations continue, and (ii) for computing the natural frequency, added soil mass should be assigned to the piling below the ground surface, based on the density of sand-water-mixture.

Tsunamis are caused by the crustal deformation in the seabottom due to earthquakes along the slope of the seabed or a volcanic eruption under water. They create violent forces and wash out effects, and could cause extensive damage to floating structures. Criteria for tsunami design require

that the bathymetry in the vicinity of the structure, as well as that in the deeper ocean areas to which the site is exposed, be examined for wave refraction characteristics which would magnify or attenuate tsunami heights. The tsunami magnitude, m, can be related to wave height, as follows:

$$h = 2^m. \qquad \ldots(10)$$

The relationship between the earthquake and tsunami magnitudes is shown in Fig. 6. A rational approach to tsunami design criteria for a particular offshore site would require that a design tsunami be selected from past records (based on the probability of occurrence during the life of the structure) for each principal direction of site exposure. The refracted waves should be examined to determine (a) resonant characteristics of bays or inlets in which the structure is located, (b) maximum wave amplitude, period, and character (breaking, non-breaking or bore), (c) maximum wave forces, and (d) scouring velocities around the base of the structure.

4.0 EXISTING TEST PLATFORMS

4.1 Introduction:

Available records of ice force measurements on structures include measurements on test and actual offshore platforms at Cook Inlet, Alaska, bridge piers in Alberta, and lighthouses in the Gulf of Bothnia and the St. Lawrence river in Canada. Wave forces have been measured on a number of test and regular platforms in the Gulf of Mexico, and the North Sea.

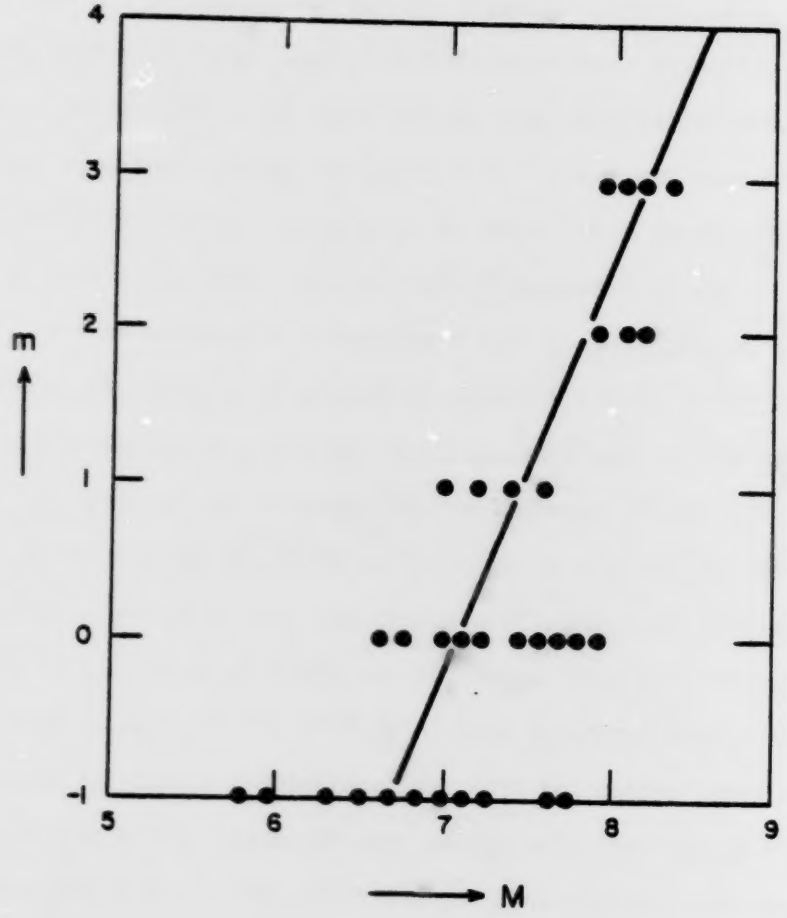


Figure 6 Relationship between Earthquake Magnitude M and Tsunami Magnitude m (Alford, et al 1969).

4.2 Ice Force Measurements:

4.2.1 Cook Inlet and Beaufort Sea

The only measurements of ice forces on offshore platforms were those made at Cook Inlet, Alaska. However the ice conditions in the Beaufort Sea are very different from those at Cook Inlet. Relatively thin annual ice floes (1.5 m thick) with very small ridges cause less severe loading than the multi-year ice ridges (35 m thick) of the Beaufort Sea. Measurements of ice forces on other types of structures such as artificial islands have been described by Croasdale (1977). Typical dimensions are 100 m diameter at the working surface, and freeboard of around 5 m to 6 m. Croasdale and Marcellus (1978) have discussed the ice and wave action on islands. Typical island design should account for constructional constraints, working area needed, ice action, wave action, and geotechnical factors.

The effect of the test structure flexibility on the ice forces measured is of considerable importance in planning any test structure. Peyton (1966a,1966b,1966c and 1968) described continuous measurements of ice forces on a test pile installation at Cook Inlet from 1964 to 1968. The ice force records were those determined from two instrumented structural devices: (i) a strain-gauged test pile driven into the ocean bottom adjacent to an existing temporary offshore drilling platform, and (ii) a field test beam hinged at both ends to the platform leg with a load cell measuring the reaction at the upper hinge, Fig. 7. For a number of structures with different ice velocities Blenkarn (1970) measured differing crushing frequencies and concluded that the ratcheting frequency is a function of ice sheet velocity, structure flexibility, and the loading rate dependence of ice sheet

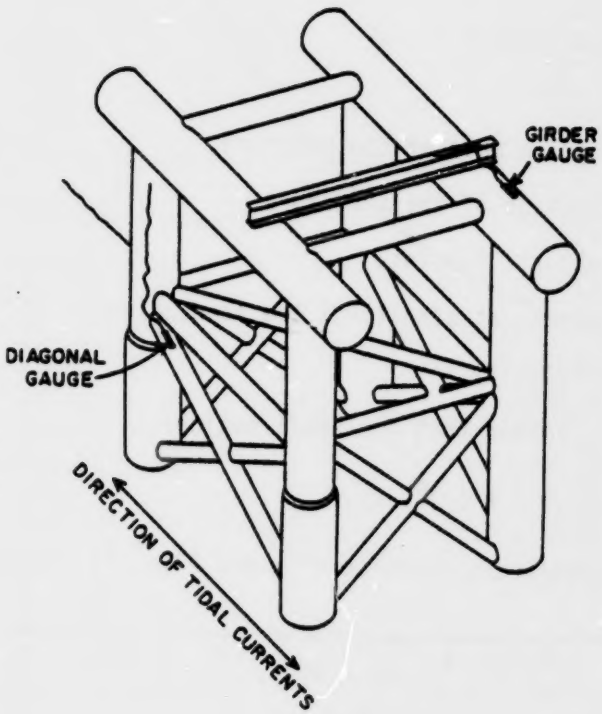


Figure 7 Strain Gauge Location - MGS 'A' Platform (Blenkarn, 1970).

crushing strength. An interesting finding was the identification of the origin of ice-induced vibration. Negative damping was observed due to decreasing ice crushing strength with increasing stress rate indicating vibration of a self-excited type.

Assuming the ice force to be a function of relative velocity of the structure and ice field velocity, the equation of motion for the structure is written in the following form:

$$(W/g)\ \ddot{x} + c\dot{x} + kx = F(v-\dot{x}) \qquad \ldots(11)$$

in which

W/g = effective mass of the platform,

x = deflection of the structure,

c = structural damping,

k = stiffness at structural ice load point,

and

v = ice velocity.

Blenkarn (1970) showed that for small motion:

$$F(v-\dot{x}) = F(v) + F'(v)\ (-\dot{x}). \qquad \ldots(12)$$

where F(v) is arbitrary function of v and F'(v) is its derivative. Hence

$$(W/g)\ \ddot{x} + (c + F')\ \dot{x} + kx = F(v). \qquad \ldots(13)$$

Assuming certain properties based on Fig. 8, Blenkarn (1970) has shown that (c + F') could be less than zero implying self-excited oscillations.

Durning (1976) and Utt et al (1976) analyzed the recorded dynamic response of the monopod platform in Cook Inlet, and simulated the response with computer models in an attempt to find the reason for ob-

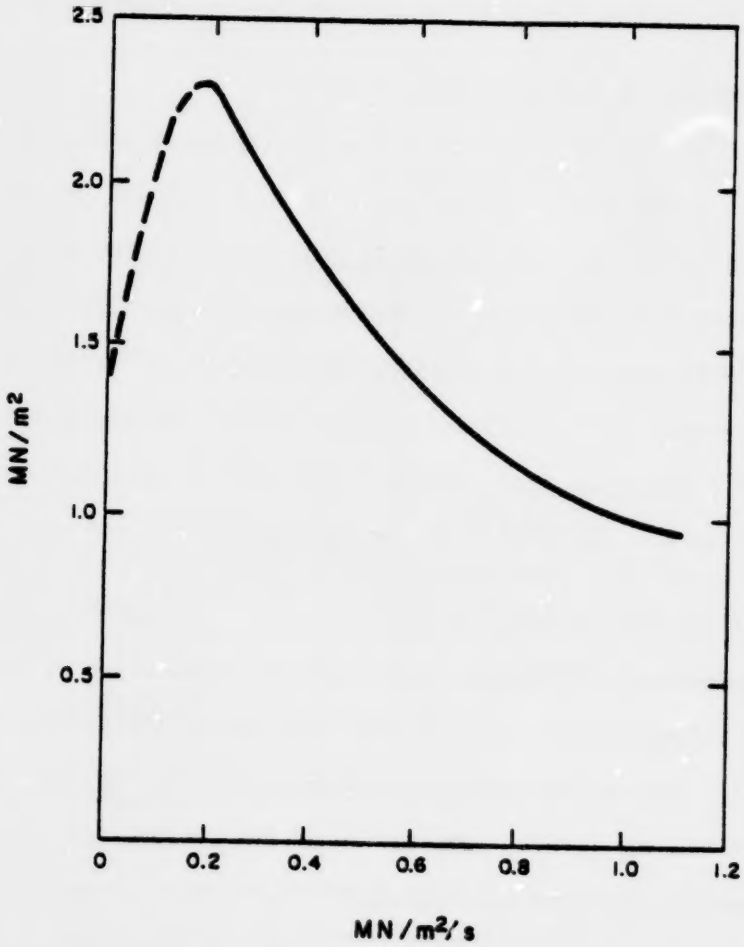


Figure 8 Crushing Strength vs Strain Rate (Blenkarn, 1970).

served changes in the fundamental frequency (structural integrity monitoring). An accelerometer system was installed to provide information on dynamic design parameters and specific data on the platform's ability to withstand strong environmental forces from earthquakes or ice floes.

4.2.2 Pembina and Battle Rivers, Alberta:

Neill (1970, 1972) carried out extensive measurements of ice forces on several bridge piers in Alberta. The work included the synchronization of the ice force recording and movie photography at one site to compare force fluctuations with the nature of ice failure. Danys (1970) and Danys and Bercha (1975) reported ice pressure measurements on navigation light piers at St. Lawrence river in Canada. Danys (1975) also described the installation of a specially instrumented light pier to measure ice forces at Lac St. Pierre. The details of the instrumentation are shown in Fig. 9.

4.2.3 Baltic Sea:

Engelbrektson (1977) has described the computation of dynamic ice loads on a lighthouse structure from recorded accelerations at orthogonal locations. The maximum measured acceleration of a typical lighthouse was 0.33 g at the top level. Computation of the dynamic ice loads was based on the assumption that the two fundamental components of motion can be distinguished from the time-history diagram of the deflection. Relatively slow motion, which is finally attenuated during the passage of an ice sheet as ice breaks against the structure, causes the "mean" ice-thrust. This component is independent of the motion of the structure and therefore by nature is of a static type. The other component is a vibrating

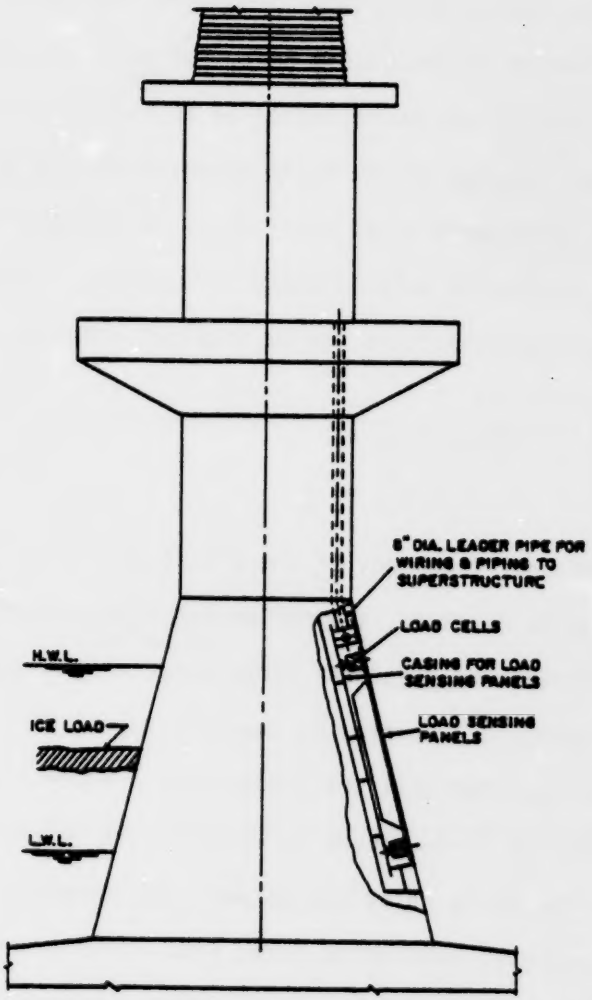


Figure 9 Arrangement of Ice Force Measuring Installation at Yamachiche Lightpier (Danys, 1975),

one which is the fluctuating part. The frequency of this fluctuating part is constant and is close to the structural fundamental frequency.

Engelbrektson indicated a simple method for ice force measurement on an offshore lighthouse in the Baltic Sea. The basic assumption is that as the internal damping can be neglected as it has little effect on the generating force. In view of the basic observations and findings of Blenkarn (1970), which were later verified by Maattanen (1975), this approach would be applicable only to heavy structures. If the damping term is neglected, the ice force can be obtained from the following expression:

$$(W/g)\ddot{x} + kx = F \qquad \ldots(14)$$

$$\text{or, } \ddot{x} + \omega^2 x = \frac{Fg}{W} \qquad \ldots(15)$$

where ω is the natural frequency of the structure. Once the recorded acceleration data is obtained, the deflection time-history is determined by double integration and the total force computed. A typical force history of a lighthouse, reported by Engelbrektson (1977) is shown in Fig. 10. It was observed that the predominant frequency of the fluctuating part of the ice force record is close to the natural force vibration. However, the basic relations between ice strength, ice thickness and drift velocity that need to be taken into account during dynamic ice structure interaction were ignored. Therefore, the application of this procedure for estimating the dynamic ice forces on structures is limited.

Based on the comments of Blenkarn (1970), on self-excited oscillations for ice-induced vibration, Maattanen (1977a, 1977b) carried out dynamic ice structure interaction studies of relatively flexible steel lighthouses suffering from severe vibrations. Instrumentation procedures were developed

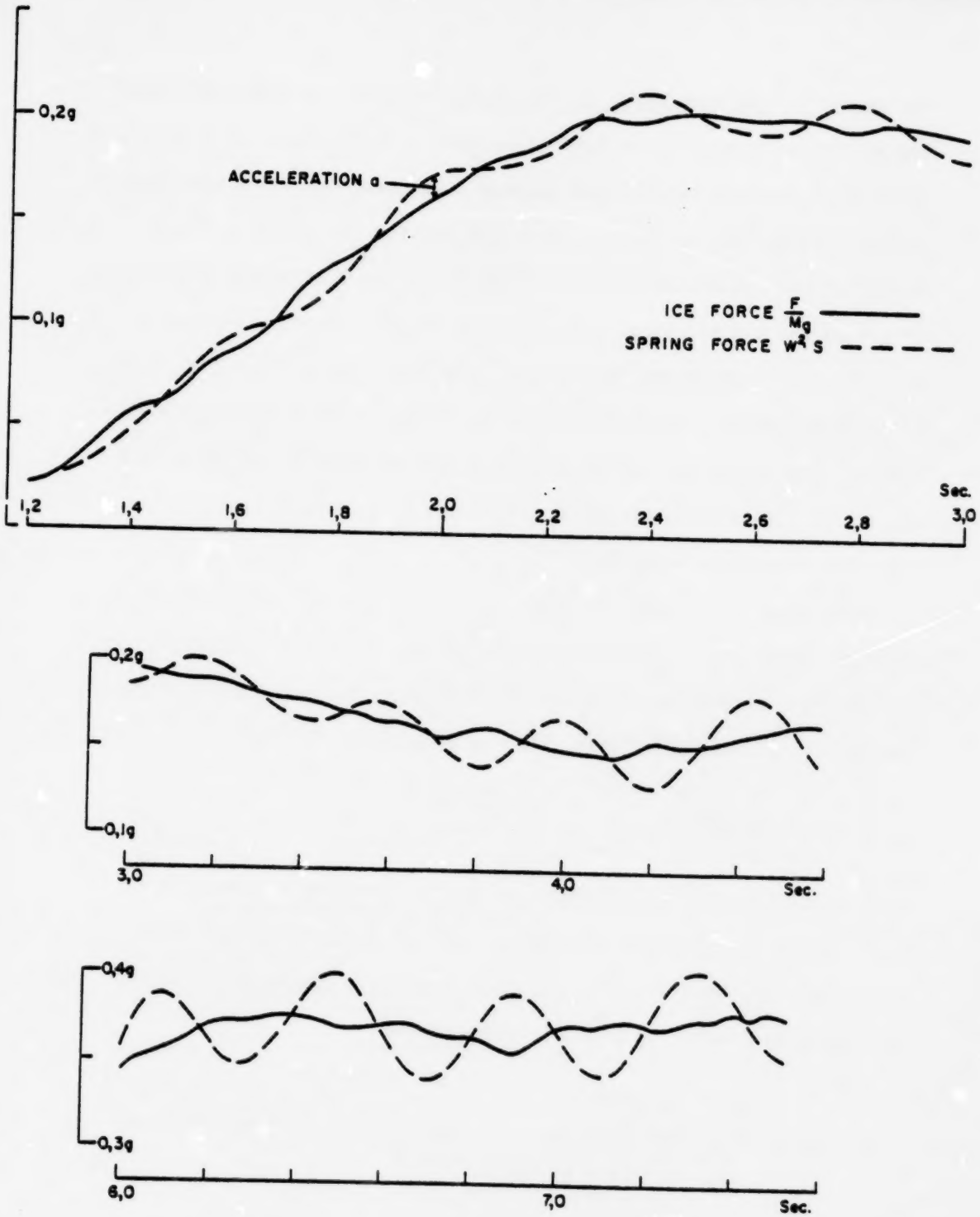


Figure 10 Force time-histories derived from the Example No. 1 records (Engelbrektson, 1977).

for measuring the forces due to ice floes and ridges on the lighthouses at the northernmost end of the Gulf of Bothnia. The maximum ice loading was found to be associated with the maximum pressure ridge. The resultant pressure ridge load was 2 to 3 times the maximum ice sheet load. Maattanen also suggested a mathematical model by relating the properties of ice to the dynamic equations of motion of the structure through an averaged ice crushing strength curve. The resulting differential equations of motion were solved based on the theory of self-excited oscillation. The comparison of the predicted and measured force plots indicates that ice-induced damping during crushing is considerably greater than pure structural damping.

Maattanen (1978) was the first to include the ice properties such as crushing strength, thickness, velocity, and structural properties to estimate the dynamic ice force record and study the ice-induced vibrations. The stress rate was calculated from the equation:

$$\dot{\sigma} = (v - \dot{x}) \frac{4\sigma_c}{\pi a} \qquad \ldots(16)$$

where σ_c is the crushing strength of ice and a is the diameter of the pile. If σ_c is assumed to depend not only on $\dot{\sigma}$, but also on θ, the polar angle of the pile, Fig. 11, then assuming a cosine distribution considered by Frederking and Gold (1971) gives,

$$\sigma_c(\dot{\sigma},\theta) = \sigma_c\ (\dot{\sigma})\ \cos\theta \qquad \ldots(17)$$

$$\text{or,}\quad \dot{\sigma} = (v - \dot{x})\ \cos^2\theta\ \frac{4\sigma_c(\dot{\sigma})}{\pi a}. \qquad \ldots(18)$$

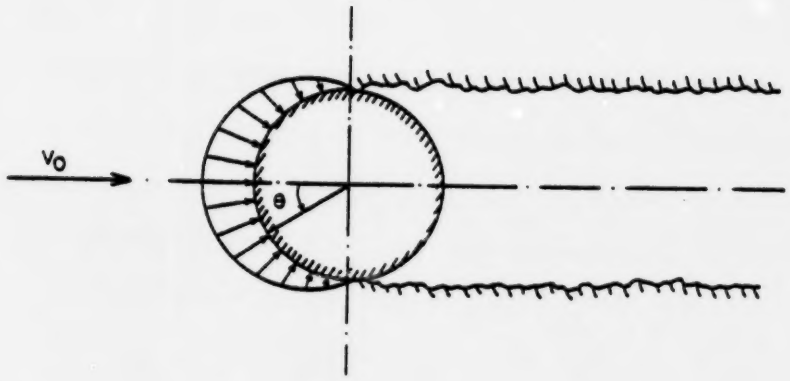


Figure 11 Ice pressure against a pile (Määttänen, 1977 (a)).

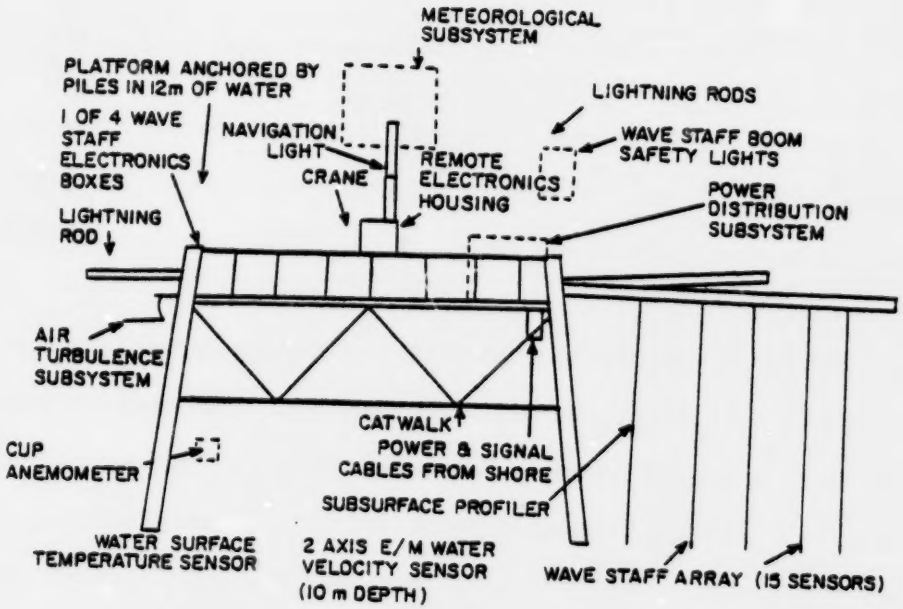


Figure 12 Wave-76 Platform with instrumentation (Roy, et al, 1978)

Piece wise linearization of the highly nonlinear crushing strength curve, Fig. 8, gives

$$\sigma_{cj} = \alpha_j + \beta_j \dot{\sigma} \qquad \ldots(19)$$

and the stress rate for the j th can be obtained in closed form as follows:

$$\dot{\sigma}(\theta) = \frac{\alpha_j}{\frac{\pi a}{4(v - \dot{x})} \cos^2 \theta \cdot \beta_j}. \qquad \ldots(20)$$

The ice force, F, is then obtained by integrating over the pile as follows:

$$F = (2h) \int_0^{\pi/2} \sigma_c(\dot{\sigma}, \theta)(\cos\theta)\, a\, d\, \theta. \qquad \ldots(21)$$

The force term which is on the right hand side of Eqn. 11 is split into two parts: static component {Fo} due to ice crushing strength with the initial constant ice velocity; and the component dependent on the change of relative velocity as $\{F_1\}$. Therefore,

$$\begin{aligned} F(v - \dot{x}) &= \{Fo\} + \{F_1\} \\ &= \{Fo\} + (\phi)\ \{\delta\} \end{aligned} \qquad \ldots(22)$$

where

$$(\phi) = (\phi(\dot{\sigma})),$$

(ϕ) is linearized and denoted as a diagonal coefficient matrix with non zero terms, ϕ_{nn}, only in the degrees of freedom associated with the ice load point n. ϕ_{nn} was evaluated by Maattanen (1977b) as follows:

$$\phi_{nn} = -8\, hd^2 \int^{\pi/2} \frac{\sigma_j \beta_j \cos^4 \theta}{(\pi d - 8\beta_j v \cos^2 \theta)^2}\, d\phi \qquad \ldots(23)$$

where the integrand is evaluated numerically.

4.3 Wave Force Measurements

While the primary aim of this report is the proposal for a test platform in the ice- and ridge-infested Beaufort Sea, experience with the test platforms built for wave force measurements both large and small, offers a great deal of useful information.

4.3.1 Gulf of Mexico

Haring and Spencer (1979) described a large scale engineering experiment, data acquisition system (DAS) installed on a Ocean Test Structure (OTS) in the Gulf of Mexico for the measurement of environmental loading primarily due to wave forces and to monitor the structure for this kind of loading. The framed platform size was 20 x 40 x 120 ft. and it was installed in 66 ft. of water in November 1976 and moved in June 1978. The OTS program was sponsored by the Exxon Production Research Company and the following organizations: American Bureau of Shipping, Amoco Production Compnay, Atlantic Richfield Company, Bureau Veritas of France, Chevron Oil Field Research Company, Det Norske Veritas, Gulf Research and Development Company, Mobil Research and Development Corporation, Nippon Kokan K.K., Penzoil Company, Shell Oil Company, Sun Protection Company, and the United Kingdom Department of Energy. The storm data is representative of typical environmental loading. The instrumentation was calibrated during the construction period and checked at a later date after the installation. Data obtained from this OTS measurement system consisted of mainly local wave forces, total base shear and moments on the structure, the well conductor forces together with the impact forces on a member above the mean water level. Responses produced by real ocean waves were represented as random vector time series and special techniques developed to analyze

these data. Applications of the measured data include evaluation of local force coefficients, total wave forces, wave induced velocity profiles, and frequency response functions. The condition of barnacle growth was monitored throughout the test program.

Finn and Young (1978) described a test on a guyed structure in 89 m of water in the Gulf of Mexico. The results obtained from test data indicate that the wave-induced dynamic response of the tower can be accurately simulated and that the spud can foundation performed satisfactorily. Based on the results, it is further shown that the guyed tower is a technically feasible and practical structure to serve as an offshore drilling and production platform.

4.3.2 Christchurch Bay Project, U.K.:

Bishop (1978) described the Christchurch Bay project which was intended to provide wave force data on a gravity type platform to fill the gap between small scale laboratory experiments and full scale site conditions. The reason for choosing the site at Christchurch Bay was that the wave conditions represent reasonably the North Sea environment but on a reduced scale. The typical instruments used in carrying out the full scale test operations were signal conditioning recorders and an on board computer. Bishop and Holmes (1978) also described the preliminary results of spectral and probabilistic analyses of wave and wave-induced loads on the main tower at Christchurch Bay. The results obtained from data show reasonable agreement with linear random wave solutions; the influence of the nonlinear drag component of the loading was also observed. Brown and Huckvale (1978) described a vibration monitoring system and the computer analysis of measured vibration spectra for the pur-

pose of using the technique as a means of determining the structural integrity of ocean structures.

4.3.3 Bedford Institute of Oceanography Test Structure

Mills (1972) described the design procedure and instrumentation set up for a small offshore structure for air-sea interaction studies and oceanographic research. The research was carried out at the Bedford Institute of Oceanography. The three-legged diagonally cross-braced structure weighing 25 tons was 65 metres in height with an equilateral triangle cross section of side 2.45 m. The structure, which rested on the seabed and was held in position by a system of guy wires, failed in a severe storm after surviving for a long period in 8 m seas.

4.3.4 Lake Ontario Research Platform

Roy, Der and Gibson (1978) described a study of wave-induced strain on a free standing rigid four-legged research platform, Fig. 12, in 12 m of water at the west end of Lake Ontario. The platform was designed based on wave loads estimated from a 6 m, 7 sec. design wave. Using the computer software STRESS, the structural analysis was carried out. A comparison was made between the calculated and the measured strains and the usefulness of the design model was evaluated on this basis. The study indicated that relatively simple modelling techniques could give a conservative results in determining the stresses for design purposes.

4.3.5 American Bureau of Standards Tower

Wojanarowski, Stainseu, and Reddy (1977) described a procedure for rational evaluation of the structural integrity of a fixed base offshore

platform based on vibration calculations, and on board vibration measurement. The SAP - IV computer code was used to carry out the analysis for the mathematical model taking into consideration the added mass, soil support conditions etc. The vibration measurements on a test framed tower, near New York Harbour, were processed by a mini-computer through a spectrum analyser to establish the correlation between computed and measured responses.

4.3.6 Imperial Oil Monocone

In view of particular relevance to ice infested waters a monocone type structure developed by Imperial Oil was tested by the Marine Dynamics and Ship Laboratory and the Hydraulic Laboratory of the National Research Council, Ottawa. The test simulated a Beaufort Sea condition for wave loading (wave height of 12 m at a water depth of 21 m - 41 m).

4.4 Ice Force Measurements

Oshima, Narita, Yashima and Tabuchi (1980) described measurements for ice forces on conical structure in the sub-Arctic and Arctic. The ice force measuring experiments included, i) laboratory tests, ii) outdoor tests using medium size field models in a lagoon and iii) actual full scale field tests on a large size tower which was specially installed in the Okhotsu Sea.

The tower dimension at the field was 5.5 m diameter at water level with a total height of 11 m. The ice thickness at the field was 30 cm to 60 cm. Primary future of ice observed in the field was of a bending nature. Measurement of the normal and axial compressive forces at the base of the tower were obtained from 8 strain gauges mounted on the side

wall. The strain signals were transmitted to the land by subsea cable.

The model tests which were conducted in an ice model basin included monocone, monopod and four legged conical structures. The Arctic ice model basin was used for these model tests and unconsolidated and semi-consolidated first year pressure ridge and raft ice were used for environmental loading.

Outdoor tests were conducted at the Saroma lagoon near Mombester using medium sized models of various ice resistant structures.

5.0 CONCEPT EVALUATION - Watt (1977)

5.1 Choice of Structural Form

Examples of the fixed-base concepts are the Steel Framed Tower, Concrete/Steel Gravity Platforms and hybrids, and other compliant structures such as the Guyed Tower. For compliant structures, such as the guyed tower, the trade off is between excursion amplitude and the restraining force. In fact, the compliant structure is not applicable to the ice force environment, particularly when large ridge forces are involved. The choice between the framed and gravity monopod concepts is conditioned by foundation considerations. The heavily over-consolidated soils, close to the mudline of the North Sea, which were created by pleistocene glaciation, can support large loads and are, therefore, well suited to gravity type foundations. This is distinct from foundations on large layers of soft, recent marine sediments which would necessitate installation of foundation support at some depth below the mudline. The advantage of 'transparency' to fluid loads on the framed structure is irrelevant to the ridge loading in the

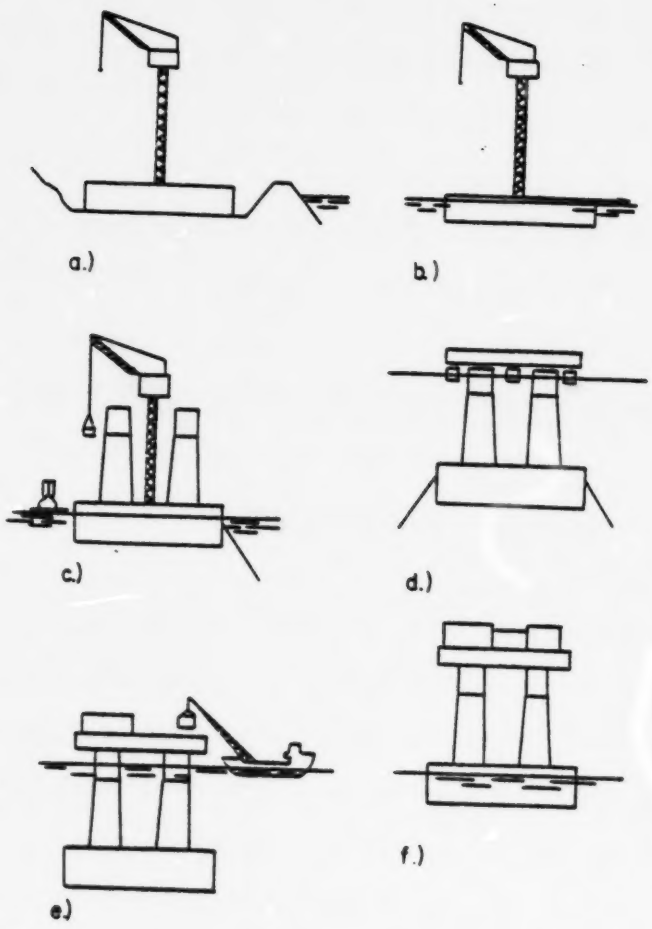
Beaufort Sea. Therefore, the only suitable fixed base concept is the gravity monopod. It eliminates the need for costly pile driving operations required for framed platforms and reduces installation time to a minimum. A typical construction scenario, shown in Fig. 13, indicates its particular relevance to harsh environment areas such as the Beaufort Sea.

Due to the very large forces that ice exerts on a gravity structure, it has been proposed that structures subjected to the ice force should be provided with sloping faces to reduce the magnitudes of ice forces. As an example, the sides of bridge piers and lighthouse pedestals subject to ice forces are often made inclined for this purpose. The reduction of forces is due to the fact that it is generally easier to fail an ice sheet by applying a vertical load that will cause it to fail in bending than by applying a horizontal load to induce direct compression and consequent crushing. Pearce and Strickland (1979) have indicated that the reduction in ice forces on an offshore structure would be greater as the inclination of the interacting force increases.

The measurement of ice forces on conical structures area is very difficult due to the complexity of the problem. Since 1959, the Marine Aids Section Section of the Canadian Coast Guard has used the following formulae proposed by Danys and Bercha (1975), for calculation of horizontal and vertical ice forces on cone-shaped lighthouses as given below:

$$H = F_x = mn_1 \beta h \sigma \qquad \ldots(24a)$$

$$V = F_z = mn_2 \beta h \sigma \qquad \ldots(24b)$$



Key a.) Construct part of base in cofferdam.
b.) Float out and moor.
c.) Complete concrete construction afloat.
d.) Float steel deck over ballasted-down structure.
e.) Load modules.
f.) Refloat to towing mode.

Figure 13 Typical construction scenario for concrete gravity platforms (Watt, 1977).

where

m = shape and contact coefficient,

$n_1 = \cos^2\alpha$,

$n_2 = \cos\alpha \sin\alpha$,

α = slope angle with the vertical,

β = width of the structure at the design water level,

h = effective thickness of the ice sheet,

and

σ = effective compressive strength of ice.

More recently, Ralston (1977) has indicated approximate techniques for calculating the ice forces on conical shaped structures based on plasticity theory, and forces due to ridges based on the 'beam on elastic foundation' concept.

Pearce and Strickland (1979) proposed a further refinement in the design concept for a conical configuration by providing the sides of the structure with a "multiangle" conical configuration as shown in Fig. 14. The sides of the structure near the waterline are given a fairly steep angle for resisting the forces from first-year type sheet ice. The sides of the strucutre, well below the waterline, are also inclined in order to keep the forces, from the deep draft multi-year ridges, below acceptable levels. The experimental investigation, which was carried out in a model test basin for both single and multiangle conical structures, indicated that the latter have the advantages of

a) providing the necessary height of the deck above the waterline,

b) minimizing the area above the waterline exposed to uniform sheets of ice,

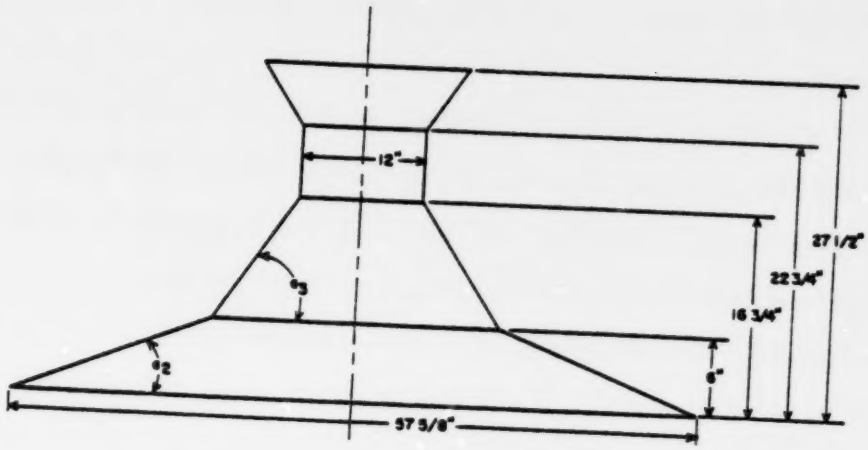


Figure 14 Multi-angle Model Cone Geometry (Pearce and Strickland, 1979)

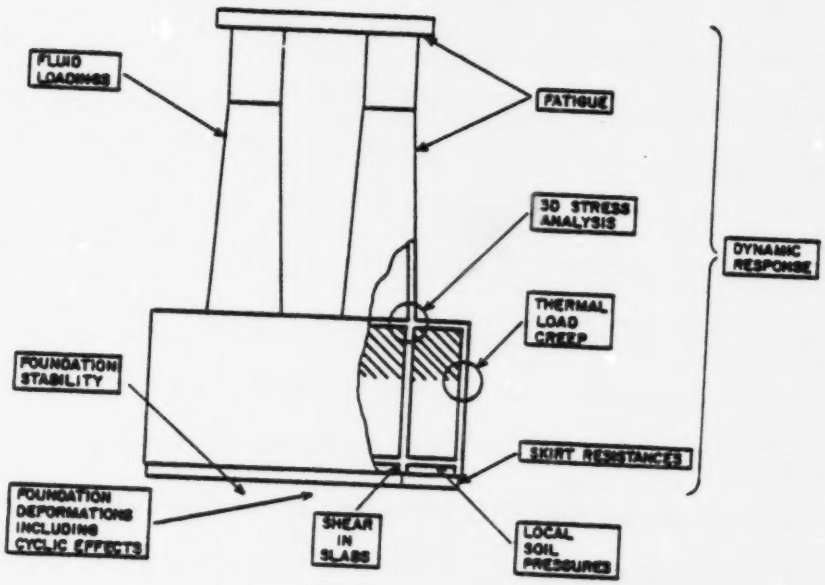


Figure 15 Current Analytical Problem Areas for Gravity Platforms (Watt, 1977).

c) a large base mat to react to the ice loads, and

d) a shallow strong conical section below the waterline to fail multi-year ridges.

A slope-sided, heated hull Arche Drill Barge suitable for depths up to 10 m has been recently proposed by Crager(1980). The barge has been indicated as "an attractive alternative to other methods such as gravel island" because of its mobility, ease of restoring the site to its original condition and less expense when used for several wells.

5.2 Materials:

Concrete and steel are both being used in the marine environment for offshore structures. A properly designed concrete mix is efficient and durable for the marine environment. Concrete has higher damping characteristics than steel; structurally prestressed concrete has better dynamic characteristics and is probably safer than its equivalent form of steel (Gerwick, Mansour, Price and Thayamballi, 1978).

Typical examples are a steel platform which has been built in Cook Inlet by Union Oil Co. and a proposed concrete concept, called Monocone, for the Arctic environment by Imperial Oil Ltd. For making the structural material corrosion-resistant, a coating mix of the type sold by International Paints (Canada) Limited has been used (Mäkinen, Lathi and Rimppi, 1975).

5.3 Analysis:

An overview of the analytical problems to be considered for an offshore gravity platform is shown in Fig. 15. The areas to be considered

are three-dimensional stress analysis, elastoplastic or viscoplastic deformation of the foundation for soil-structure interaction, and steady and transient flow problems for fluid-structure interaction.

The basic analysis procedure is based on the static stiffness method because the dominating environmental load from the ice sheet and the multi-year ridge is a static one. For other kinds of loads, such as waves and earthquake excitations, a proper dynamic response analysis should be carried out; the transient nature of the environmental loads, and the coupling between the fluid, structure, and soil systems make the problem very challenging. General procedures have been suggested by Moan et al (1975) for the dynamic response of the structure to wave inputs.

Provided that the remote boundary conditions can be adequately characterized, it is possible to model both structure and foundation using the finite element method. A two-stage approach is commonly used; a dynamic finite element analysis for the foundation is carried out first to determine the frequency dependent foundation impedances. These are used for a deterministic response history analysis, based on either a time domain or a frequency domain approach. As the environmental loading conditions are uncertain and random in nature, most investigators appear to favour the use of a stochastic approach where the loading condition is characterized by a spectral density function. This approach is most suitable for a gravity platform analysis due to a linear transfer function compared to a nonlinear one for a steel jacket platform due to the drag form.

5.4 Foundation and Soil Conditions:

Foundations on the seabed differ from those on land in several important respects, apart from the sheer size of structures designed for depths of 65 m. Significant differences are as follows:

a) soils are generally saturated,

b) subsoil exploration is much more difficult and expensive,

c) sediments affected by movement due to waves and currents are often sorted into regions having nearly homogeneous surface deposits (sand in particular, is often nearly uniform in grain size and may be densely packed),

d) waves induce eccentric and inclined dynamic loading (in particular, large oscillatory shear forces can occur),

e) currents associated with waves are frequently strong enough to cause scour, particularly around large monoliths.

A systematic review of the foundation engineering of the North Sea gravity structures installed to date, was presented by the Schjetne, Anderson and Lauritzsen (1979). Their state-of-the-art report included recommended analyses of permanent and cyclic foundation deformations, dynamic behaviour, foundation stability, skirt penetration resistance and base contact stresses, potential piping and erosion problems and stresses in drill wells and risers.

Bjerrum (1973) has shown that gravity structures founded on densely packed sand can support steady shear forces of the magnitudes likely to be encountered (e.g. at Ekofisk), but that repeated stress reversals can cause a rise in pressure under undrained conditions that will, in time, reduce the

factor of safety during a storm. However, there would be a drop in pore pressure following the storm. This effect is greatest in sands and silts of low porosity, particularly in dense thick layers of materials.

5.4.1 Stability Analysis

For wave action different forms of the limiting equilibrium method are used, eg. Morgenstern and Price (1965), Janbu (1973) and in Lauritzsin and Schjetne (1976), several load combinations are considered. No drainage of excess pore water pressures is assumed (undrained) and various failure patterns such as sliding along the structure-soil interface or cutting the shallow weak layers, and slip circles are checked. The undrained shear stress values over the parts of the potential failure surface are determined from laboratory and field tests.

5.4.2 Skirt Penetration

The most important part of the geotechnical analysis of a gravity platform is the prediction of skirt penetration resistance.

Lunne and St. John (1979) developed the following empirical formulae for thin-walled steel skirts and dowels

$$q_t(d) = K_t\, q_R(d) \quad = \text{skirt lip resistance} \qquad \ldots(25a)$$

$$f(z) = k_f\, q_R(Z) \quad = \text{skirt skin friction} \qquad \ldots(25b)$$

where:

k_t and k_f = empirical coefficients varying with soil type

$q_R(z)$ = Dutch cone penetration resistance (CPT) at depth z

d = penetration depth.

Numerical analysis techniques considering the inhomogeneity of typical installation sites and variation in skirt profiles, could be based on a Lagrangian formulation with a general dilatant constitutive law for predicting the safe resistance value. Under the extreme environmental loading condition, it is generally necessary to use a full three-dimensional bearing capacity theory for a stability analysis of an offshore gravity foundation problem. Hansen (1970) has developed such a theory which is the most popular one and being used to date. Recently, Vaughan (1976) developed finite element techniques for the analysis of the stability of large gravity structures taking into account the variation in shear strength with depth.

Hydraulic instability must be considered in the stability analysis accounting for the effective change in surcharge caused by the transient pressure wave. An approximate analysis, based on the simplified finite difference formulation treating the problem to be a consolidation one, has been reported by Chan (1976). Cyclic degradation of the foundation, due to change in both stiffness and shear strength of soils, should be taken into account in designing the gravity foundation.

5.5 Construction Scenario

A typical construction scenario of a gravity platform is shown in Fig. 13. Part construction of the base caisson in a dry dock, is followed by flooding the dock, towing the rest of the structure to a protected deep water construction site and mooring it. The rest of the structure is then completed with slip forming of the caisson walls and legs. The prefabricated deck structure (up to 10,000 tonnes) is loaded on the

platform legs. The installation of the deck is done by ballasting down the concrete structure to enable the floating of the deck into position over the legs. The concrete caisson at this stage is partly flooded and therefore subjected to large net ambient pressures. During the deck mounting parts of the caisson are subjected to differential pressures, of 1.3 to 1.6 MPa, which is about three hundred times the design distributed line load in a typical building. A typical caisson structural system is shown in Fig. 16, a) assembly of cylindrical cells, b) cellular box with straight walls acting as a system supporting a pressure resistant skin. The ambient pressure is resisted by cylindrical surfaces, in membrane compression.

Recently Erde, Kjekstad and Brylawski (1979) summarized the experience gained during the installation of concrete gravity structures in the North Sea. Their study included a comparison of the design data with actually recorded field measurements during the installation phase.

5.6 Instrumentation

The direct benefit from a well instrumented offshore structure can be a) better structural design criteria, b) control during operation, c) fatigue information, d) improvement in safety, and e) great cost savings. Typical fixed platforms, which were instrumented to measure the environmental forces and structural response to environmental forces are shown in Figs. 17, 18, and 19. Instrumentation to measure response to ice loading on an offshore platform at Cook Inlet, Alaska is shown in Fig. 20. Ice-induced vibration of the platform was measured with accele-

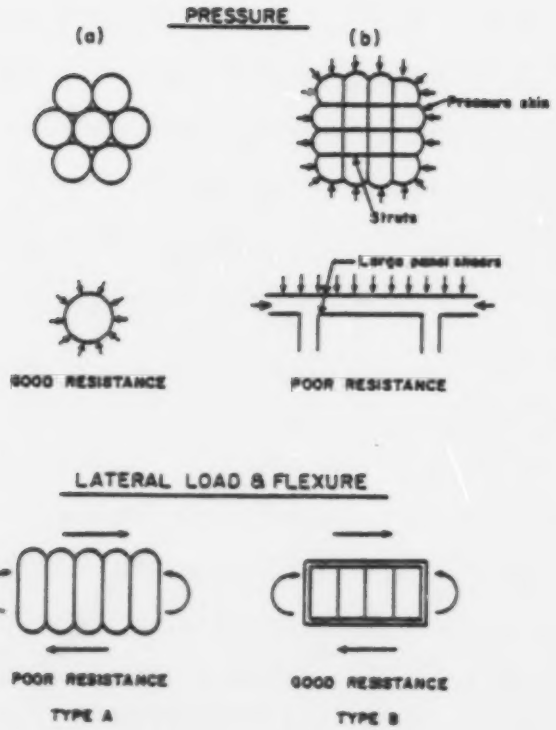


Figure 16 Typical Caisson Structural Systems (Watt, 1977).

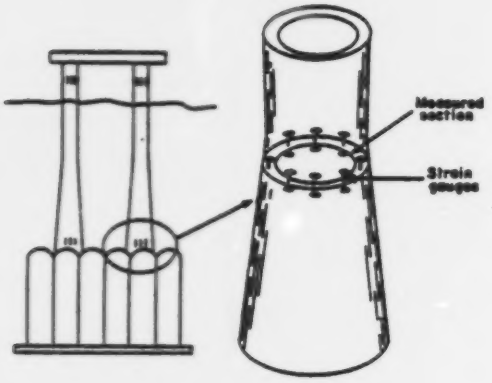


Figure 17 Installation of strain gauges on concrete platforms (Bourgeios, 1977).

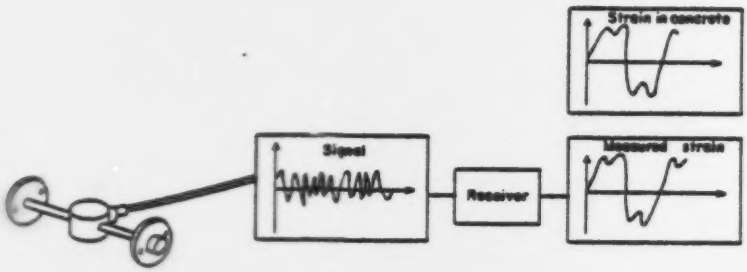


Figure 18 Principle of strain gauges (Bourgeois, 1977).

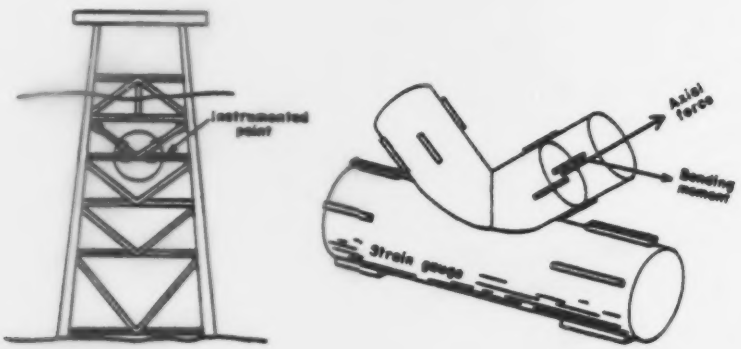


Figure 19 Installation of strain gauges on steel platform (Bourgeois, 1977).

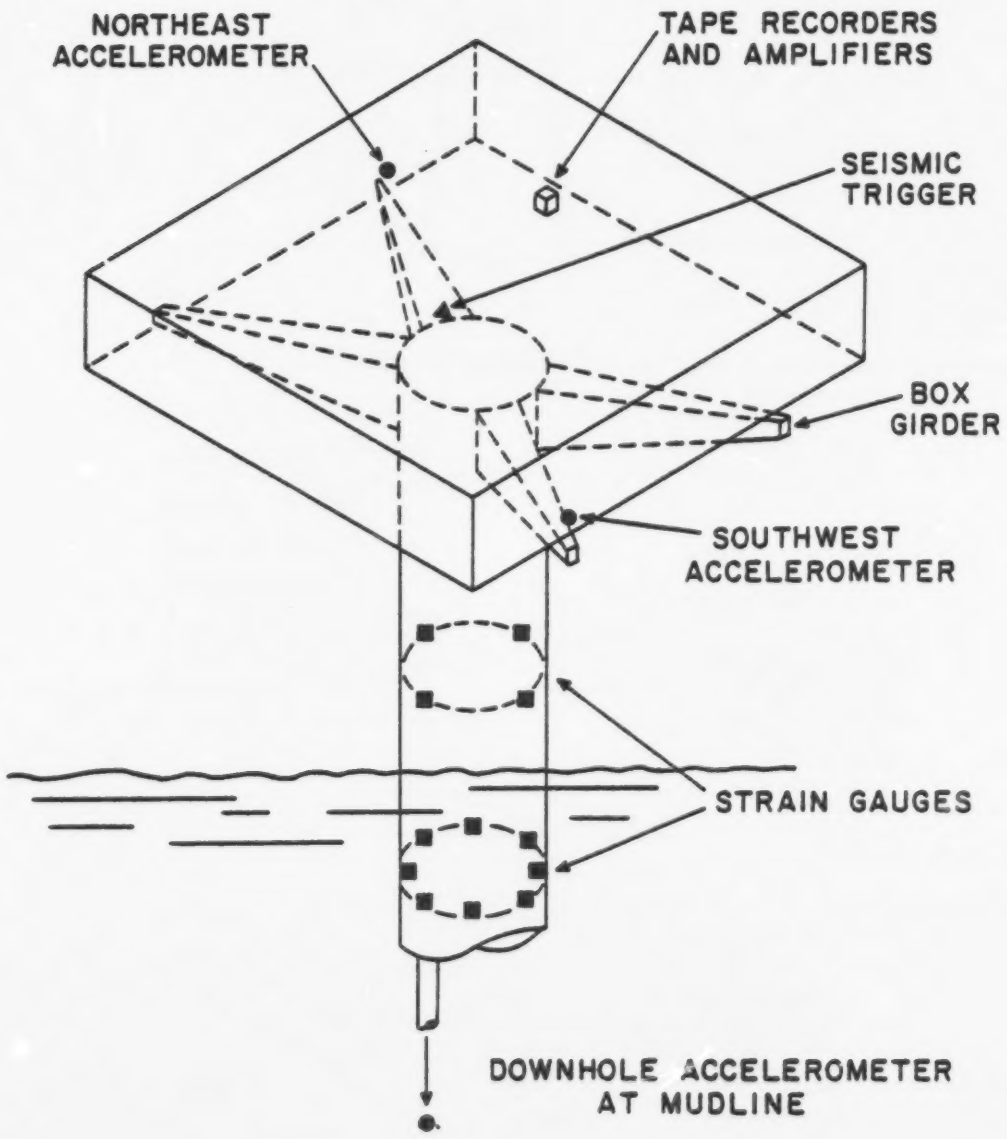


Figure 20 Monopod vibration monitoring system (Durning, et al, 1976).

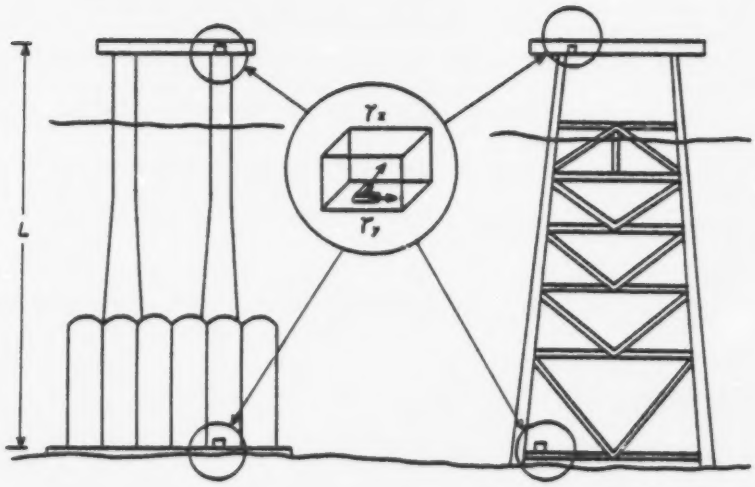


Figure 21 Installation of simplified accelerometer system (Bourgeois, et al 1977).

rometers. The use of a double linear accelerometer unit, orthogonally set-up in the horizontal plane, is shown in Fig. 21 for typical locations in the structure. Once the accelerations and displacements are recorded, the ice force record can be determined by the method of Engelbrektson (1977).

The signals which are recorded in a measuring process can be divided according to the speed of their variations: slow and fast channels. The recording is on magnetic tape in digital or analogue form according to the type of signal. Bourgeois and Leroy (1977) indicated that slow channel signals can be recorded in digital form and fast channels in analogue form. Details of the instrumentation techniques have been summarized by Bourgeois and Leroy (1977), and also by Geminder and Pomonik (1975).

Other information that is of considerable importance is the effect of fatigue on the structure.

6.0 PROPOSED DESIGN AND COST ESTIMATE FOR A TEST STRUCTURE IN THE BEAUFORT SEA

6.1 Environment

The Canadian Beaufort Sea oil exploration activity extends some sixty miles offshore from the Mackenzie Delta and the Tuktoyaktuk Peninsula (Fig. 22). From mid-October to late June, this zone is affected by ice, but is normally ice free during the remainder of the year. Ice distributions, as described by Stenning and Schumann (1979), in the Canadian Beaufort Sea can be categorized into three zones: the landfast zone, the shear or transition zone, and the polar pack. The proposed bottom-founded platform that has to be capable of measuring full scale ice force data in the Beaufort Sea, will be in a depth ranging from 46 m to 75 m. The

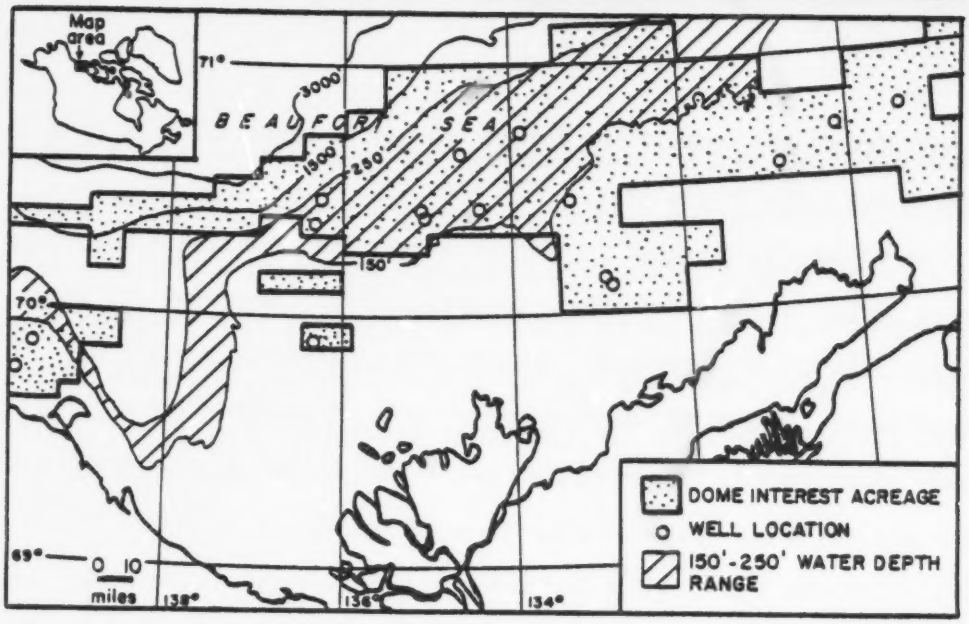


Figure 22 (Stenning, et al, 1979).

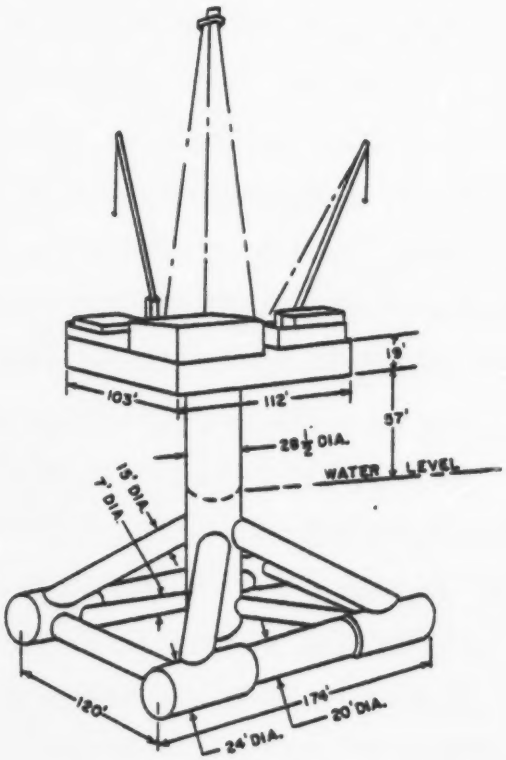


Figure 23(a) Monopod platform in Cook Inlet, Alaska (Blenkarn, 1970).

Beaufort seabed soil conditions change from dense sands to loose silts in the east and clays in the west.

The structures designed in the transition and pack area should consider a number of ice conditions: compressive failure of the ice sheet, rafting, pile up, and pressure ridges. Several other parameters have to be considered: the effect of low temperatures on construction material behaviour under impact, the short open water season, and the extremely high labour and equipment costs.

Air temperatures range from -50°C to +30°C with a minimum water temperature of -1.67°C. Wind speed is predominantly 15 to 25 km/hr with a maximum of 80 to 100 km/hr. Fog is common offhsore during the summer, especially along the edge of pack ice. Wind generated waves are less than 10 m with a predominant typical period of 10 secs. The tidal range is only 0.3 m and current speeds are 4.8 km/hr and 2.4 km/hr at the surface and bottom respectively. The largest design loads are from the effect of sea ice. The most severe load is postulated by assuming a 35 m multi-year ridge frozen into a 3 m thick multi-year ice sheet as shown in Fig. 2. Because of the conical nature of the superstructure, the ice sheet and the ridges will fail in flexure rather than in crushing. Once the ridge fails, the ice is forced past the structure subjecting it to large loads. Typical earthquake loading to be considered is 8.3% of gravitational acceleration according to Zone 3 of the National Building Code of Canada for seismic analysis in the Beaufort Sea.

The structure has to be capable of continuously breaking up the advancing ice. Break up can be in one or more failure modes; crushing,

buckling, bending, and shear. Typical unconfined compression test values for these failure modes are: crushing 2,100 kPa; bending 700 kPa; and shear 420 kPa. The loads due to the impingment of the moving ice sheet will depend on the particular failure mode. Confined compression tests reported by Kivisild and Iyer (1976) have given three times larger values. However unconfined compressive values are reasonable for most offshore environments.

6.2 Recommendations for Selection of Ice Resisting Structures

The general design philosophy should take into account minimization of the ice load, fabrication, and pre-assembly of the structure. Considering the numerous alternatives, and based on previous experience, both framed and gravity type structures seem feasible in ice-infested water. Because of the shallow water depth (less than 183 m), and the ice environment compliant structures cannot be considered. The framed gravity type would be feasible only in 'thin ice' regions with comparatively small ridges, like Cook Inlet, Alaska. The advantages of the concrete type gravity test structure compared to the pile-type, are: 1) large bottom weight making it very stable provided soil conditions are satisfactory to carry these heavy loads, 2) provision to refloat for movement to other locations, and most importantly 3) its rigidity.

A recently developed monopod concept permits easy adjustment of height to accommodate varying water depths. It consists of a base, a single supporting leg and the ice breaker module as a superstructure. The advantage of fabricating the total facility as a single unit is that it can be towed while floating upright. The gross geometry of the structure will be dictated by the need to minimize the ice loads and to mobilize sufficient

strength on the sea floor. Based on the monopod concept, a typical monocone for the Arctic water has been developed by Dome Petroleum Ltd. (Stenning et al 1979). This structural type is made up of three major components: the doughnut-shaped base, the bottle-shaped super-structure, and the jack-up production deck. The superstructure may be described in terms of an annular bottom section, a conical midsection, a cylindrical shaft, and a jacking ring.

Three types of proposed gravity structures are shown in Fig. 23. The first one is a steel gravity monopod, Fig. 23(a), built in Cook Inlet, Alaska, by Union Oil Company. The foundation is a pile-supported one with an approximate cost of $145 million based on the quantity estimate. The second concept is a concrete gravity monopod designed by the Sea Tank Co. (Laeroix and Pliskin, 1970), Fig. 23(b). With a caisson foundation, the approximate cost is $7 million. The third concept is a monocone (two alternate types are shown in Figs. 23(c) and 23(d); the costs involved would be of the order of 25 and 60 million dollars. Typical load calculations for the second concept, Fig. 23(b), are shown in the section on numerical analysis.

For a firm clay soil in the Beaufort Sea with a cohesive shear strength of 120 kPa, it is anticipated that a base with outside and inside diameters of 105 m and 30 m, respectively, would be adequate for foundation stability, Fig. 24. A skirt penetration of 1.0 m below the ground would be enough to prevent sliding. For stability analysis to assure an adequate factor of safety against soil failure, the governing load case should be considered assuming simultaneous seismic excitation and operating ice loads. The

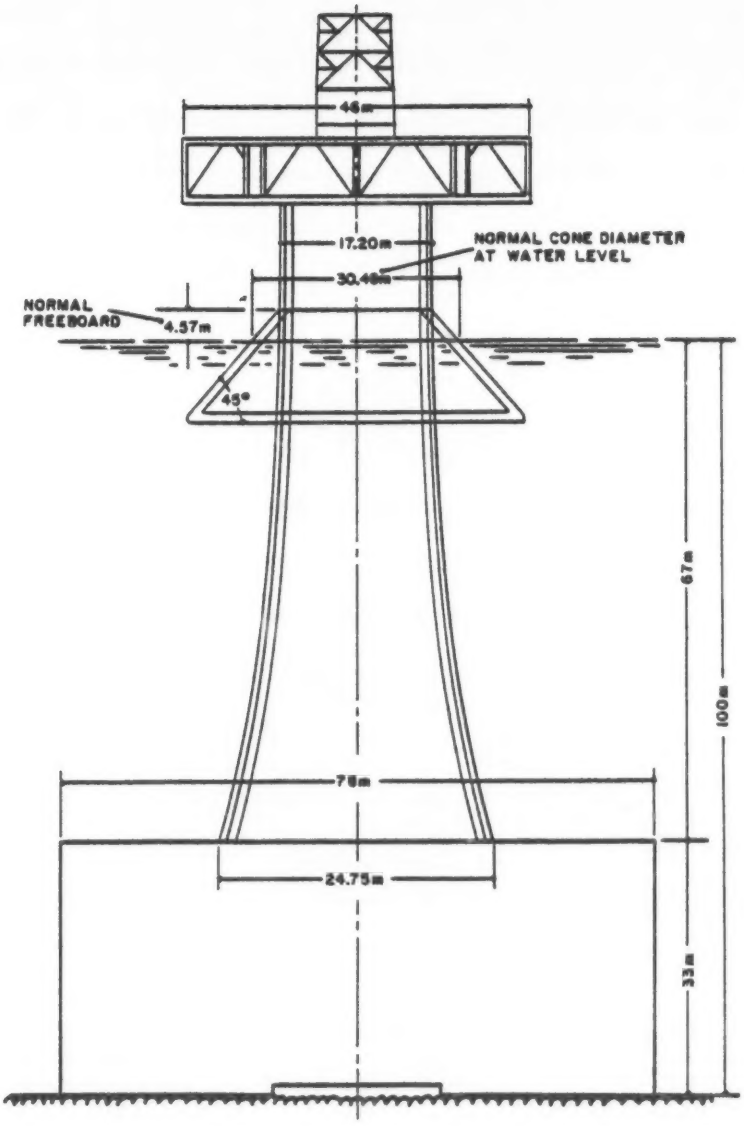


Figure 23(b) Gravity Monopod - Sea Tank Co. (Laeroix et al, 1970).

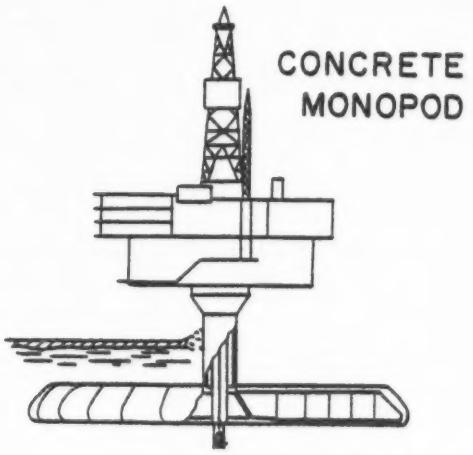


Figure 23(c) (Croasdale, 1977).

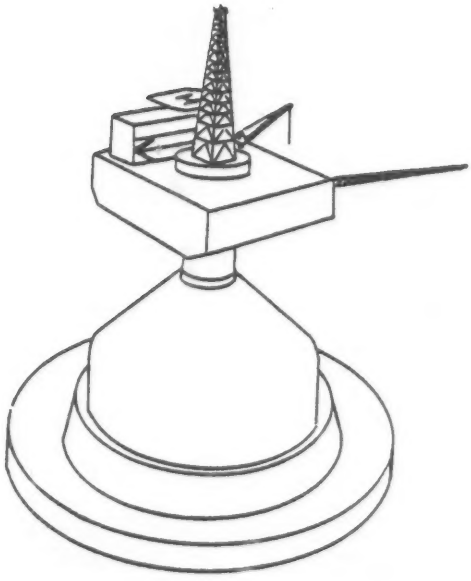


Figure 23(d) (Stenning et al, 1979).

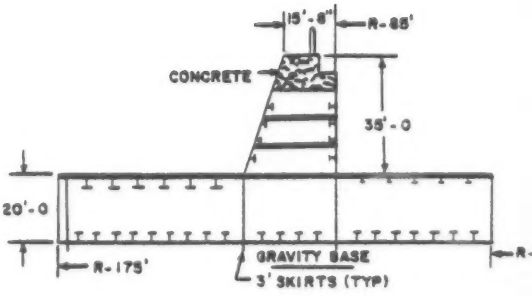


Figure 24 (Jaz rawi et al, 1977).

limiting stability condition is based on a sliding failure induced by the applied horizontal loads.

Gerwick and Karp (1979) have described the use of icebreaker experience in the evaluation of sea ice forces on structures. As an icebreaker cuts across a varied sequence of sheet and ridge forms, it integrates the forces that could act on a fixed structure in a similar ice environment. An energy approach, based on a three-dimensional failure criterion, is indicated as a practical means for correlation of force components on the vessel to this achrgon a fixed offshore structure. The scarcity of literature on upward ice breaking, restricts the transposition of information to a conventional conical shape.

Another concept for measuring ice forces would be to move an icebreaker with the model of a monopod type structure mounted on it through an ice field. When the icebreaker cuts through the field, the force exerted on the monopod would be measured through a transducer or load cell. The difficulties which may arise in measuring the ice forces, compared to that of a fixed structure, are due to the relative velocity of the icebreaker and the ice field. Unless the icebreaker is moving relatively slowly, e.g. a speed close to the velocity of the ice field (1 m/sec.), the forces measured would be very conservative. Slow speeds for the icebreaker may also cause entrapment.

7.0 TYPICAL DESIGN LOAD CALCULATION FOR A GRAVITY MONOPOD - FIG. 23(b)

The gravity monopod considered is a modification of the concept proposed by the Sea Tank Co. (Laeroix and Pliskin, 1970) for a production platform in the North Sea. The modification consists of an addition of

a conical module for inducing the bending mode of failure in ice sheets.

7.1 Computation of Ice Loading

7.1.1 Ice Sheets

Ralston (1977) developed the following expressions for ice loading on conical structures based on the theory of plasticity:

$$R_H = (A_1\sigma_f t^2 + A_2\gamma_w tD^2 + A_3\gamma_w t\ (D^2 - D_T^2))\ A_4 \qquad ...(26)$$

and

$$R_V = B_1 R_H + B_2\gamma_w t\ (D - D_T^2) \qquad ...(27)$$

where

R_H = Horizontal force on the cone,

R_V = Vertical force,

$\gamma_w = \rho_g$ Weight density of water,

D = Diameter at water level where ice is acting,

D_T = Top diameter,

t = Ice sheet thickness

and

σ_f = Flexural strength

A_1, A_2, A_3, A_4, B_1 and B_2 = coefficients to be obtained from the graph of Ralston as given in Fig. 25.

Horizontal load at water level:

$$R_H = (A_1\sigma_f t^2 + A_2\ \gamma_w tD^2 + A_3\gamma_w t(D^2 - D_T^2))\ A_4 \qquad ...(28)$$

where

$\sigma_f = 6.89 \times 10^5$ N/m^2, A1 = 1.75,

t = 2.0 m, A2 = 0.12,

$\gamma_w = 9794$ N/m^3, A3 = 0.30,

D = 30.49 m, A4 = 1.30,

D_T = 17.20 m, B1 = 1.00,

μ = 0.15, B2 = 0.035,

$\alpha = 45^o$

Substituting these values in Equation 28 gives

$$R_H = 13.954 \text{ KN}$$

$$R_V = B_1 R_H + B_2 \gamma_w t \ (D^2 - D_T^2) \qquad \ldots(29)$$

$$= 18{,}308 \text{ kN}$$

Moment at the base level (due to horizontal ice load alone):

$$R_H \times 58.0 \text{ m} = 13{,}954 \times 58.0 \text{ kN-m}$$

$$= 809{,}332 \text{ kN-m}$$

7.1.2 Ridges - (Ralston 1977) and Figs. 26-28

The vertical force calculations based on the two significant events that can occur during ridge failure by imapct against the conical structure, Fig. 26, may be described as: initial crack formation and hinge crack formation. The crack formation is similar to the simultaneous failure of two semi-infinite floating ice beams under vertical loading.

The vertical forces on the core are:

$$R_V^{\infty} = \frac{4I\sigma_f}{y_t^{\ell}}, \text{ (initial crack)} \qquad \ldots(30)$$

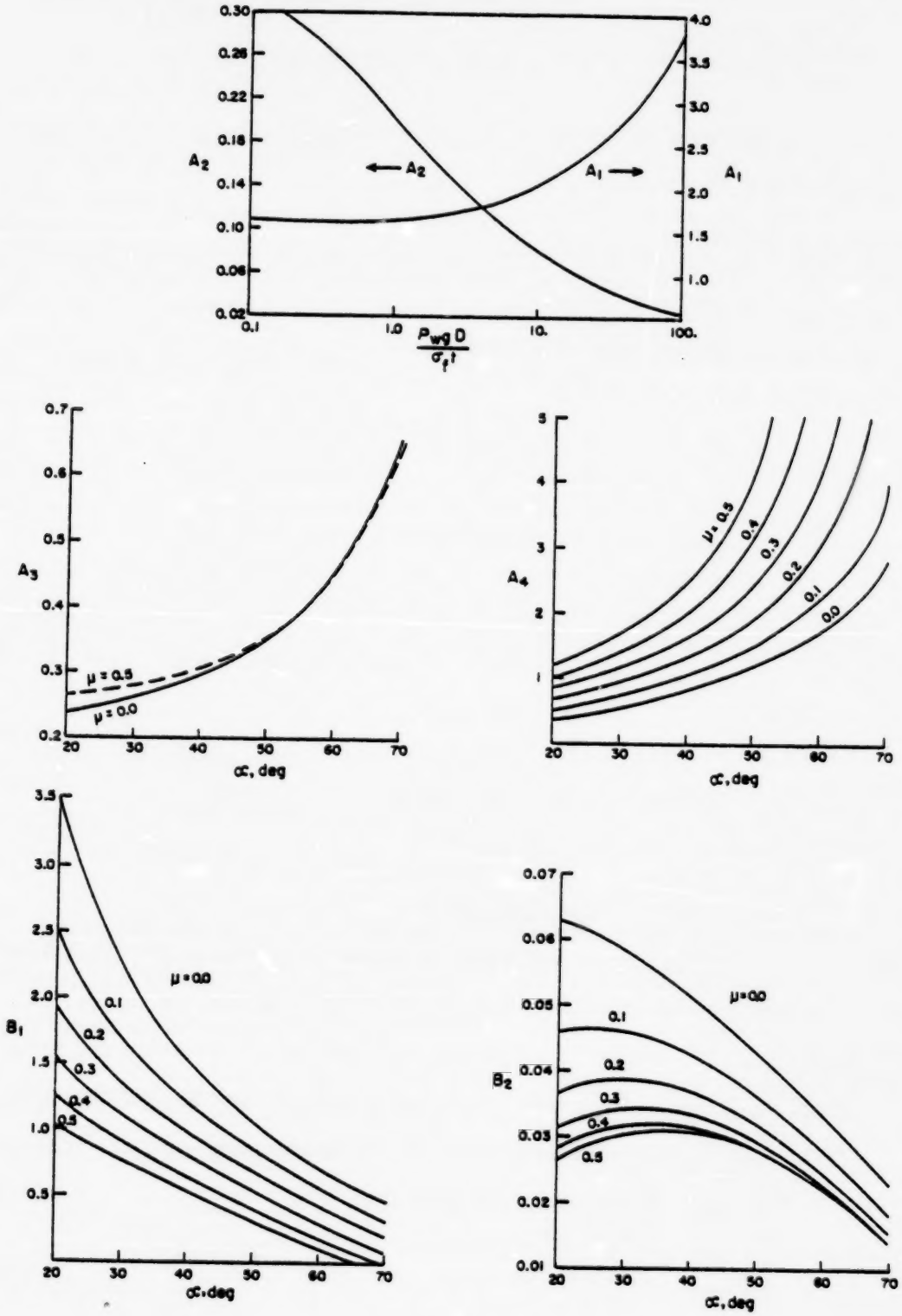


Figure 25 Ice force coefficients for plastic analysis (Ralston, 1977).

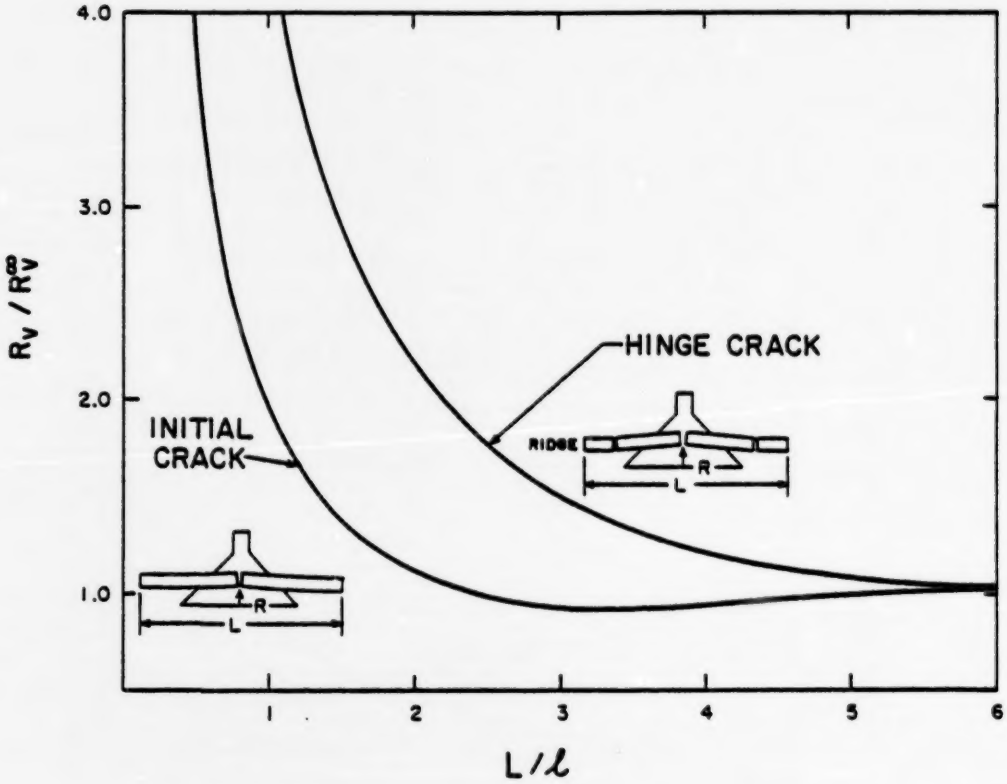


Figure 26 Elastic interpretation of initial crack and hinge crack. Forces for multi-year ridges failing against conical structures (Ralston, 1977).

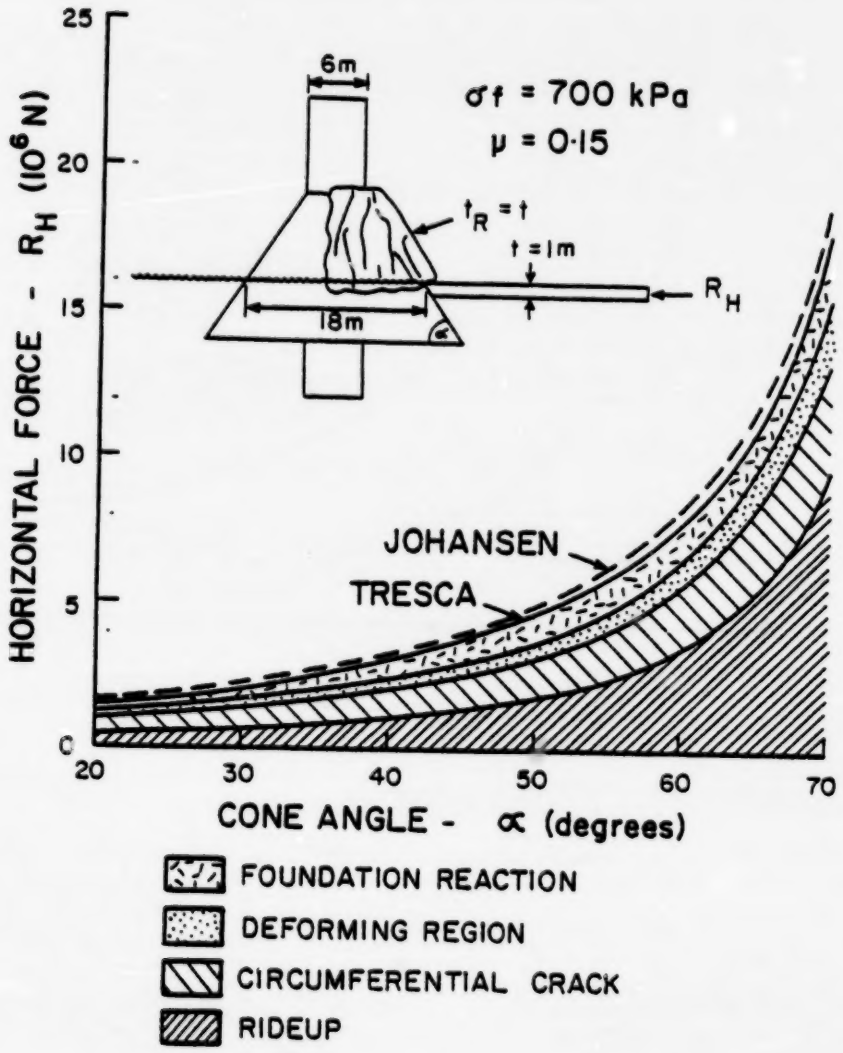


Figure 27 Ice Forces on an upward breaking cone (Ralston 1979).

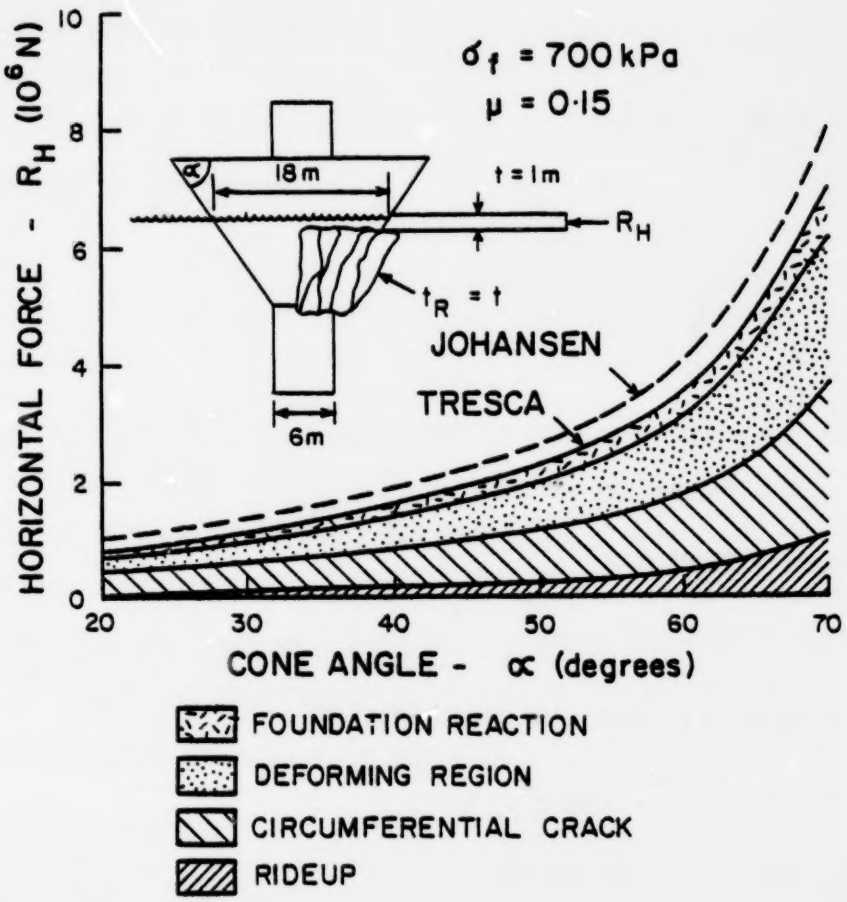


Figure 28 Ice Forces on a Downward-Breaking Cone (Ralston, 1979)..

and

$$R_V^{\infty} = \frac{6.20 I \sigma_f}{y_b \ell} \quad \text{(hinge crack)} \qquad \ldots(31)$$

where

∞ = superscript qualifying the semi-infinite nature of the ice beams

σ_f = flexural strength of the ridge,

I = cross section moment of inertia,

y_t & y_b = distance from the neutral axis and the top and bottom of the ridge respectively,

ℓ = ridge characteristic length when considered to be a beam on elastic foundation,

$$= \sqrt[4]{\frac{4EI}{k}} \qquad \ldots(32)$$

in which

E = Young's modulus of the ridge,

and

k = foundation modulus.

Thickness = 35.05 m

R_V^{∞} = initial crack = $\frac{4 I \sigma_f}{y_t \ell}$

I = moment of inertia of cross-section

$$= \frac{1 \times 35.05^3}{12} = 3588 \text{ m}^4,$$

ℓ = characteristic length,

$$= \sqrt[4]{\frac{4EI}{k}} = \sqrt[4]{\frac{4 \times 5.52 \times 10^9 \times 3588}{9798}} = 300 \text{ m}$$

in which k is the foundation modulus = γ_w

R_V^{∞} = Initial crack = 1,876 K N/m,

$$R_V = \text{Hinged crack} = \frac{6.20\ I\ \sigma_f}{y_b^{\ell}}$$

= 2907 kN/m = 88634 kN on the structure.

By using a resolution factor, given by Danys and Bercha (1975), the vertical force due to ridge load (hinged crack), F_2, is related to the transformed horizontal load, F_x as follows:

$$F_x = \zeta F_z, \qquad \ldots(33)$$

where

$$\zeta = \frac{\mu \cos \alpha + \sin \alpha}{\cos \alpha - \mu \sin \alpha} \qquad \ldots(34)$$

in which

μ = coefficient of fritction,

and

α = cone angle.

$\therefore$ $\zeta = 1.36 \times 2$ for $\mu = .15$ and $\alpha = 45^{o}$

Hence $F_x = 1.36 \times 2907.0$ kN/m = 3.953 kN/m

for a diameter of 30.49 m at waterline, the total horizontal force from a pressure ridge, 35.05 m deep, would be

= 3,953 x 30.49 = 120,542 kN.

Equivalent moment at the base = 120,542 x 58

= 6.99×10^6 kN-m

7.2 Wind Loading - (Det Norske Veritas, 1974, 1975)

Assuming a wind speed of 100 mph (100 knots)

$$V_{10} = \frac{100 \times 1760 \times 3}{60 \times 60} = 146.67 \text{ ft./sec.} \quad (44 \text{ m/sec.})$$

V_h = 146.67 x 1.19 = 174.29 ft./sec.

= 53.12 m/sec.

Dynamic force (quasi-static)

$$F_w = 0.0625\ C_s A\ V_h^2$$

where C_s = shape coefficient.

= 62,675 KP (Kgf) = 614.215. kN

Total moment at base

= 614.2 kN x 68 = 41,752 kN-m

7.3 Wave Force (Including inertia force effects), (Det Norske Veritas, 1974, 1975)

Wave Height, H = 11.0 m

Wave Period, T = 8.5 sec

Wave Length, $L_o = \frac{g}{2\pi} T^2 = 1.56\ (8.5)^2 = 112.71$ m

Diameter D of structure at M.W.L. = 17.20 m

Wave Number, $K = \frac{2\pi}{L} = 0.0557$

and

Depth of Water, d = 58.0 m

$$\frac{\eta_o}{H} = \frac{1}{2} + \frac{1}{4}\left(\frac{\pi H}{L}\right)^2 = \frac{1}{2} + \frac{1}{4}\left(\frac{3.14 \times 11.0}{112.71}\right)^2 = 0.52$$

The effective lever arms for the drag and inertia force are given as follows:

l_D = lever arm for drag force = $d(\frac{Kd - 1/2}{Kd})$ = 49.02 m

and l_I = lever arm for inertia force = $d(\frac{Kd - 1}{Kd})$ = 40.05 m

checking $\frac{d}{L_o} = \frac{58}{112.71} = 0.51$

since $\frac{d}{L_o} > 0.51$ deep water condition applies, and $L = L_o$.

Hence $\tanh \frac{2\pi d}{L} \rightarrow 1.0$.

Deep water approximation for u and $\dot{u}$ may then be used:

$$u = \frac{\pi H}{T} \exp \frac{2\pi\xi}{L_o} \cos 2\pi(\frac{x}{L} - \frac{t}{T})$$

and

$$\dot{u} = \frac{2\pi^2 H}{T^2} \exp \frac{2\pi\xi}{L_o} \sin 2\pi(\frac{x}{L} - \frac{t}{T}) \qquad \ldots(36)$$

Next it is necessary to determine C_M and C_D.

For a circular cross-section $C_M = 2.0$

The maximum Reynolds number is

$$0.9 \times 10^6 \times \frac{\pi H}{T} \exp \frac{2\pi \times 0}{20} \times 17.20$$

$$= 62.90 \times 10^6.$$

Based on this Reynolds number, a value of $C_D = 0.7$ is taken

Hence
$$F_D = \frac{1}{2} \rho C_D D \int_{-d}^{o} u |u| \, d\xi \qquad \ldots(37a)$$

$$= 0.5 \times 1025 \times 0.7 \times 17.20 \int_{-58}^{o} u |u| \, d\xi$$

$$= 6170.50 \times (\frac{\pi H}{T})^2 \int_{-58}^{o} \{\exp \frac{2\pi\xi}{L_o}\}^2 \cos 2\pi(\frac{x}{L} - \frac{t}{T}) / \cos 2\pi(\frac{x}{L} - \frac{t}{T}) \, d\xi$$

$$= 914197.94 \left[1 - e^{-(6.46)} \right] \cos^2 2\pi\left(\frac{x}{L} - \frac{t}{T}\right)$$

$$\therefore F_D = 912{,}734 \cos^2 2\pi\left(\frac{x}{L} - \frac{t}{T}\right) \qquad \ldots(37b)$$

And

$$F_I = \rho \times C_M \times \pi \times \frac{D^2}{4} \times \frac{2\pi^2 H}{T^2} \int_{-d}^{0} \exp \frac{2\pi\xi}{L_o} \sin 2\pi\left(\frac{x}{L} - \frac{t}{T}\right) d\xi \qquad \ldots(38a)$$

$$= 1025 \times 2.0 \times 3.14 \frac{(17.20)}{4} \times \frac{2 \times \pi^2 \times 11}{(8.5)^2} \int_{-d}^{0} \exp \frac{2\pi\xi}{L_o} \, d\xi \cdot \sin 2\pi\left(\frac{x}{L} - \frac{t}{T}\right)$$

$$= 24{,}641{,}362 \sin 2\pi\left(\frac{x}{L} - \frac{t}{T}\right) \qquad \ldots(38b)$$

It is interesting to note that the peak inertial force is quite high as compared to the peak drag force. A result of this nature could have been predicted by the two criteria indicated below:

$$\frac{D}{L_o} = \frac{17.20}{112.71} = 0.15 \qquad \text{i.e. } \frac{D}{L_o} < 0.2;$$

Hence, use of the Morison equation is valid

$$\frac{D}{W} = \frac{17.20}{\tanh \frac{2\pi d}{L}}$$

where $\tanh\left(\frac{2\pi d}{L_o}\right) \rightarrow \frac{2\pi d}{L_o}$ for deep water wave theory.

$$\therefore \frac{D}{W} = \frac{17.20 \times 112.71}{2 \times 3.14 \times 58} = 5.32 \text{ i.e. } \frac{D}{W} > 0.2,$$ hence inertia is increasingly predominant.

The drag and inertia forces have now been independently calculated at

$$F_D = 912{,}734 \cos^2 2\pi\left(\frac{x}{L} - \frac{t}{T}\right) \quad \ldots 39(a)$$

and

$$F_I = 24{,}641{,}362 \sin 2\pi\left(\frac{x}{L} - \frac{t}{T}\right) \quad \ldots 39(b)$$

Let A_D and A_I represent the amplitude of the drag and inertia forces respectively.

Then $F = A_D \cos^2 k(x - ct) + A_I \sin K(x - ct)$

where $K = \frac{2\pi}{L}$ = wave number ... 40

For a maximum, $\frac{\partial F}{\partial x} = 0$.

$$\frac{-1}{2} A_D \times 2K \sin 2K(x - ct) + K A_I \cos K(x - ct) = 0.$$

i.e. $\sin K(x - ct) = \frac{A_I}{2 A_D}$

If $A_I \geq 2 A_D$, the peak force always occurs at $K(x - ct) = 90^o$ and is equal to A_I. If $A_I < 2 A_D$, the peak force will occur at some sharp angle between 0^o and 90^o as given by the above formula.

In the example $\frac{A_I}{2 A_D} = \frac{24{,}641{,}362}{2 \times 9.2{,}734} = 13.50$

$\therefore$ peak force = A_I = 24,641,362 N

Moment at the base = 24.65 MN x 40.05 m = 9.88×10^8 Nm

7.4 Seismic Force

According to the Uniform Building Code, the criteria for earthquake load give the follow equation for base shear, V,

$V = Z\ KCW$

where

Z = zone factor,

K = type of structure (offshore),

C = flexibility value,

and

W = weight of the structure

For $T = 1.0$ sec.,

$V = 441{,}0$ kN

Moment = 441 kN x 89 m = 0.39×10^5 kNm

7.5 Loading Combinations

Wave and Wind Forces (Case I)

Ridge and Wind Forces (Case II).

Sectional property of the tower at base, treating it as an unreinforced section:

I = Moment of inertia = 3022 m^4.

Case I: Moment = 9.88×10^5 kN-M + 0.42×10^5 kN-M

Case II: Moment = 6.99×10^6 kN-M + 0.42×10^6 kN-M

$= 7.032 \times 10^6$ kN-M

$\therefore$ For Case II: Check bending stress in compression

Structural Integrity at the Base:

$$\sigma_c = \frac{My}{I} \qquad \ldots(41)$$

$$= \frac{7.032 \times 10^6 \times 8.60\ Nm^2}{3022\ m^4}$$

$$= 0.02\ MN/m^2 < 44.13\ MN/m^2 \text{ (permissible)}$$

$\therefore$ Sectional property is O.K.

7.6 FOUNDATION DESIGN: Det Norske Veritas (1974, 1975)

Case II Ridge and Wind Forces:

Vertical force = 88,634 kN

Horizontal force = P_H = 120,542 + 614 = 121,156 kN

Moment = 7.032×10^6 kNm

Assume cohesive strength of soil c = 15 kN/m^2; $\phi = 30^o$

Computation of self weight:

Assuming the caisson to be filled with water,

weight of caisson (neglecting self weight) = $78 \times 78 \times 33 \times 9.81 \times \frac{10^3}{10^3}$

= 1,969,573 kN

Weight of cylindrical tower = $\frac{\pi}{4} \times (20.5^2 - 17.5^2) \times 50 \times 150 \times 0.1571$

= 105,495 kN

Total self weight = 2.075×10^6 kN

Vertical load including pressure ridge load = $P_V = 2.16 \times 10^6$ kN

e = eccentricity = $\frac{7.032 \times 10^6}{2.16 \times 10^6} = 3.25$ m

$\frac{e}{B} = 0.042 < \frac{1}{6}$

∴ This is compressive over the entire area.

When the friction angle $\phi \neq 0$

$$q_d = \tfrac{1}{2} \gamma' B' N_\gamma s_\gamma i_\gamma + (p' + c \cot \phi) N_q s_q d_q i_q - c \cot \phi \qquad \ldots(42)$$

q_d = design bearing capacity,

γ' = effective bulk density of soil = 18.85 kN/m^3,

B' = effective foundation width = 71.50 m,

p' = effective overburden pressure at base level = 0.0,

c = undrained shear strength = 15 kN/m^2,

N_γ = 15.07
& N_q = 18.40 } bearing capacity factors,

and

$s_q = 1 + i_q \frac{B'}{L'} \sin \phi$
$s_\gamma = 1 - 0.4\ i\ \partial\frac{B'}{L'}$ } shape factors,

in which

$$i_q = \left[1 - \frac{0.5\gamma_f\ P_H}{\gamma_f P_v + Ac \cot \phi}\right]^5, \quad = 0.8741 \qquad \ldots(43)$$

$$i_\gamma = \left[1 - \frac{0.7_{\gamma'}\ P_H}{\gamma_f P_v + Ac \cot \phi}\right]^5, \quad = 0.8274 \qquad \ldots(44)$$

and

A = base area = 71.5 x 78

γ_f = load factor = 1.2

$\therefore$ $S_q = 1 + 0.8741 \times \frac{71-5}{78} \times \sin 30^o = 1.4006$,

$S_\gamma = 1 - 0.40 \times 0.8274 \times \frac{71-5}{78} = 0.6966$,

d_q = depth factor

$= 1 + 2 \frac{D}{B'} \tan \phi (1 - \sin \phi)^2$ for foundations at a depth $D < B'$

(assuming D = 1.0 m)

$= 1 + 2 \times \frac{1.0}{71.5} \times \tan 30 (1 - \sin 30)^2$

$= 1.004$

Substituting in Eqn. (42)

$q_d = 6{,}414.89 \text{ kN/m}^2$

For stability

$$\frac{P_v \gamma_f}{A} \leq q_d \qquad \ldots(46)$$

Substituting the values in Eqn. (46)

$$\frac{2.16 \times 10^6 \times 1.2}{7.15 \times 78.0 \times 100} = 464.77 < 6416.89 \text{ kN/m}^2$$

Hence the foundation is stable

8.0 SUMMARY AND CONCLUSIONS

An attempt has been made to give an overview of the problem of design of COTS. A review of the relevant literature on environmental forces, with particular reference to ridge loading and existing test platforms, is followed up by a concept evaluation. The gravity monopod is indicated as the most feasible structure for the Beaufort Sea conditions (based on water depth, type of ice zone, soil conditions, mate-

rials, speed of installation etc;). In passing it is interesting to mention the observation of Schjetne, Anderson, and Lauritzsen (1979) that "gravity structures may safely be installed in offshore areas with less favourable platform foundation conditions than so far encountered in the North Sea."

The write-up on preliminary analysis includes choice of the structural material, static and dynamic analyses and numerical techniques. The other topics considered are foundation and soil conditions, installation problems and stability analyses, construction problems and instrumentation. This is followed up by a section on proposed design and cost estimates for a suggested test structure which is basically an adaptation of production platforms proposed for the North Sea and Arctic locations. The principal modification of the North Sea structure is the addition of a conical module to induce the bending failure in ice, in contradistinction to crushing - Fig. 23(b), The Arctic Structure is a Monocone, Figs. 23(c) and 23(d).

The numerical illustration is for the 'Conical Module Concept' in view of the availability of the structural dimensions. The design is based on simple formulae from the mechanics of materials and references to the Norwegian offshore codes Det Norske Veritas. While the caisson type foundation of the monopod is suitable for the firm clay soil reported by Stenning and Schumann (1979), pile foundations may be required for softer soils as shown in Fig. 23(a) for the Cook Inlet. The costs have been estimated in a very simplistic manner based on material weights.

While it would first appear that the icebreaker application is the least expensive concept for ice force measurements, the relative velocity of the icebreaker is a limiting factor, slow speeds are associated

with the risk of entrapment and high speeds with dynamic forces. Detailed investigation is therefore needed to explore this.

From the preliminary design calculation, it appears that the ridge load together with the wind force could be the principal environmental load for determining the stresses in the structure or at the foundation level. Stenning and Schumann (1979) pointed out that the presence of ice islands is unlikely to pose a significant threat to production platforms in the sea and suggested the feasibility of a mechanism to disconnect the superstructure from the base if required. At this stage, the authors are unable to agree with the above comments regarding the threat of ice islands. Scour due to ridges could create a serious problem for the foundation.

Installation time would be restricted only to the summer period. The paucity of environmental data with respect to seismicity and ridges would necessitate artificial generation of these forces. Parmerter and Coon (1972) indicate a method for simulating the ridge formations which is to be developed further. Model tests in a test basin need to be carried out for the proper evaluation of the loading.

The procedure described by Ralston (1977) used an elastic, perfectly plastic model for a floating ice sheet. Recently Ralston (1979) has generalized this procedure and evaluated "the need to develop more precise descriptions of ice bending strength". Numerical comparison of the upward and downward-breaking inverted cones, Fig. 27 and Fig. 28, indicates an obvious advantage for downward-breaking geometry with ice force reduction of about 50% for ice sheet loads. This also induces an upward force on the structure. As Ralston pointed out "some of the

downward-breaking advantages may be lost if ice ridges and rubble jam between the conical surface and the sea floor in shallow water applications". In the downward breaking concept the bending produces tension at the top which is at a lower temperature than the ice-water interface; this is opposite to that for upward breaking. These were considered by Ralston (1979). Further studies are therefore required to incorporate the inverted cone module in the design of COTS.

The ridge forces computed based on overall ridge thickness would increase the cost of the structure enormously. In fact, it is not necessary to design a structure for a worst ridge condition which may be a 'once-in-a-hundred-years' occurrence. The design for conventional ridge thicknesses in conjunction with a blasting procedure for the rare occurrence of a severe ridge would be an optimum one, particularly for the case of a test structure.

REFERENCES

Alford, J.L., Housner, G.W. and Martel, R.R., 1964. Spectrum Analysis of Strong-Motion Earthquakes. Technical Report, Calif. Inst. of Technology, Pasadena, Calif., August.

American Petroleum Institute Code, API, USA., 1976.

Arya, S.P.S., 1973, Contribution of Form Drag on Pressure Ridges to the Air Stress on Arctic Ice. J. of Geophysical Research, Vol. 78, pp. 7092-7099.

Asgard, P.M. and Dean, R.G., 1969. Wave Forces - Data Analysis and Engineering Calculation Method, Proc. Offshore Tech. Conf., OTC 1008, Houston, Texas, pp. 95-106

Babu, P.V.T. and Reddy, D.V., 1977. Existing Methodologies in the Design and Analysis of Offshore Floating Nuclear Power Plants. proc. International Conf. on Structural Mech. in Reactor Technology (SMIRT) San Francisco, Calif., August, J 1(a) pp: 1 - 19.

Banke, E.G., Smith, S.D. and Anderson, R.J., 1976, Recent Measurements of Wind Stress on Arctic Sea Ice. J. of the Fisheries Research Board of Canada, Vol. 53, No. 10, pp. 2307-2317.

Bishop, J.R., 1978. Description of the U.K. Christchurch Bay Wave Force Project and Some Initial Results - Part 1. Proc. European Offshore Petroleum Conference and Exhibition, Paper No: EUR23, August, England., pp. 177-186.

Bishop, J.R. and Holmes, P., 1978. Description of the U.K. Christchurch Bay Wave Force Project and Some Initial Results - Part 2. Proc. European Offshore Petroleum Conference and Exhibition, Paper No: EUR23A, August, England, pp: 187-192

Bjerrum, L., 1973, Geotechnical Problems Involved in Foundations of Structures in the North Sea. Geotechnique, Vol. 23, pp. 319-358.

Blenkarn, K.A., 1970, Measurement and Analysis of Ice Forces on Cook Inlet Structures. Proc. Offshore Technology Conf. OTC 1261 pp. 365-378.

Bourgeois, D. and Leroy, J.P., 1977. Geophysical and Mechanical Platform Instrumentation. Proc. Design and Construction of Offshore Structures, London, England, pp. 9 - 21.

Bratteland, E., Leviseth and Torset, O., 1972. Deep Water Moored Semisubmersible Platform - Theory and Model Tests, Proc. 2nd Int. Conf. on Port and Ocean Engineering under Arctic Conditions (POAC). Univ. of Iceland, pp. 227-296.

Bretschneider, C.L., 1969. Wave and Wind Loads, Section 12: Handbook of Ocean and Underwater Engineering McGraw-Hill Book Company, New York 12.4-12.7.

British Ship Research Association, Dept. of Energy. A Critical Evaluation of the Data on Wave Force Coefficients, Contract Report No. N.278, Wallsend Research Station, Wallsend, U.K., August, 1976.

Brown, D.R. and Hockvale, S.A., 1978. Monitoring and Structural Response of Steel Offshore Platform. Proc. European Offshore Petroleum Conference and Exhibition, Paper No: EUR80, August, England, pp. 169-178.

Brunn, P., Houbm, O.B., Rye, H. and Saetre, H.J., 1976. Wind and Wave and Current Data for Offshore Operations, Proc. Conf. Interocean '76, Dusseldoff, Germany, (June 1), 242-253.

Chan, K.C., 1976. A Preliminary Study of One Dimensional Flow Through Saturated Soils, Caused by Wave Loadings. Report to Ove Arup and Partners, May, England.

Crager, B. 1980. Arctic Drill Barge, Proposed for Bearfort Sea. Oil and Gas Journal, February, pp. 107 - 111.

Croasdale, K.R., 1977, Ice Engineering for Offshore Petroleum Exploration in Canada, Proc. Fourth POAC, Memorial University of Newfoundland, St. John's, Canada., pp; 1 - 32.

Croasdale, K.R. and Marcellus, R.H., 1978. Ice and Wave Section on Artificial Islands on the Beaufort Sea. Canadian Journal of Civil Eng., Vol. 5 (1), March, pp. 98-113.

Danys, J.V., 1970. Development and Installation of Apparatus to Measure Ice Forces. NRC Technical Memorandum No. 101, Workshop on the Action of Ice on Structures. Ottawa, pp. 71-72.

Danys, J.V., 1971. Effect of Cone Shaped Structures on Impact Forces of Ice Floes. Proc. First POAC, Trondheim, Norway, Vol. 1, pp. 609-620.

Danys, J.V., 1972. Effect of Ice Forces on some Isolated Structures in the St. Lawrence River. Proc. - IAHR Symp. Ice and its Action on Hydraulic Structures, Leningrad, U.S.S.R., pp. 229-231.

Danys, J.V., 1975. Offshore Installations to Measure Ice Forces on the Lightpiers in Lac. St. Pierre. Reprint IXth Int. Conf. on Lighthouses and other Aids, Ottawa, pp: 1 - 24.

Danys, J.V. and Bercha, F.G., 1975 Third. Determination of Ice Forces on a Conical Offshore Structure. Proc. POAC, Univ. of Alaska, Fairbanks, 5 pp.

Det Norske Veritas. 1974. Rules for the Design, Construction and Inspection of Fixed Offshore Structures. Det Norske Veritas Publications, Oslo, Norway, 79 pages.

Det Norske Veritas. 1975. Rules for the Construction and Classification of Mobile Offshore Units. Det Norske Veritas Publications, Oslo, Norway, 98 pages.

Draper, K., 1970. Environmental Conditions. Proc. Symp. Offshore Drilling Rigs. Roy. Inst. Nav. Arch., London.

Durning, P.J. and Engle, D., 1976, Vibration Instrumentation System Measures: An Offshore Platform's Response to Dynamic Loads. Proc. ASCE (EMD) Conf. Los Angeles, California, pp: 143 - 158.

Eide, O, Kjekstad, O. and Brylawski, E. 1979. Installation of Concrete Gravity Structures in the North Sea. Marine Geotechnology, Vol. 3, No. 4, pp. 315 - 368.

Engelbrektson, A., 1977. Dynamic Ice Force on a Lighthouse Structure. Proc. Fourth Intl. Conf. POAC, Memorial University of Newfoundland, St. John's, Nfld., September 26-30., pp. 654-663.

Federation International de la Precontrainte. Recommendations for the Design and Construction of Concrete Sea Structures, Wexham Springs, Slough, 1974.

Finn, L.D. and Young, K.E., 1978. Field Test of a Guyed Tower. Proc. Offshore Tech. Conf., OTC 3131, Houston, Texas., pp. 681-688.

Frederking, R. and Gold, L., 1971. Ice Forces on an Isolated Circular Pile. Proc. POAC Tech. Univ. Norway, Trondheim, Norway.

Geminder, R. and Pomonik, G., 1975. Instrumentation for Offshore Structures. Proc. Third POAC, Univ. of Alaska, USA., pp. 839-856.

Gerwick, B.C., 1970, Design and Construction Procedures for Proposed Arctic Offshore Structures. Proc. Offshore Tech. Conf. OTC 1260, Houston, Texas pp. 365-378, V. 2.

Gerwick, B. C., and Karp, L. B. 1979. Icebreaker Experience As A Guide to Sea Ice Forces on Structures. Proc. Civil Engineering in the Oceans. Vol. 2. pp. 622-637. September, San Francisco.

Gerwick, B.C., Mansom, A., Price, E. and Thayamballi, A., 1978. Evaluation of Technical Feasibility and Safety of 300 m Lory LPG Storage and Transport Vessel. Seminar on Concrete Ships, Proc. FIP Eighth Congress, London, Part (4), unpublished discussion

Hansen, J. Brinch, 1970. A Revised and Extended Formula for Bearing Capacity. Geotekniske Institute, Kbenhavm Bulletin, 28, pp. 5-11.

Haring, R., and Spencer, L.P. 1979. The Ocean Test Structure Data Base. Proc. Civil Engineering in the Oceans. Vol. 2, pp. 669-683. San Francisco, Calif., September, U.S.A.

Hasselmann, K.K., 1973. JONSWAP: Measurement of Wind Wave Growth and Swell Decay during the Joint North Sea Wave Project. Hamburg, Deutsches Hydrographisches Institut.

Hibler, W.D., Weeks, W.F. and Mock, S.J., 1972, Statistical Aspects of Sea Ice Ridge Distributions. J. Geophysical Research, Vol. 77, pp. 5954-5970.

Hogben, N. and Standing, R.G., 1974. Wave Loads on Large Bodies, Proc. Symp. Dynamics of Marine Vehicles and Structures in Waves. University College, London, April.

Hooft, J.P., 1971. Distribution of Wave Forces on Structural Parts of Ocean Structures. Proc. Symp. on Offshore Hydrodynamics, Wageningen, Netherlands, Paper No. I, August.

Hudson, D.E., 1965. Response Spectrum Techniques in Engineering Seismology. Proceedings, First World Conf. on Earthquake Engineering, Berkeley, Calif., June, 4-1, 4-12.

Janbu, N.L., Grande and Eggeruide, K. 1976. Effective Stress Stability Analysis for Gravity Structures. Proc. First Int. Conf. on Behaviour of Offshore Structures., Trondheim, pp. 449-466.

Jazrawi, W. and Khanna, J.K., 1977. MONOCONE - A Mobile Gravity Platform for the Arctic Offshore. Proc. Fourth POAC, Memorial University of Newfoundland, St. John's, Canada, pp: 170 - 184.

Kirillov, A.A., 1957. Calculation of Hummockness in Determining Ice Volume. Problems of the Arctic, Vol. 2, pp. 53-58. Translated from Russian by M. Slessers, U.S. Naval Oceanographic Office, Washington, D.C.

Kivisild, H. and Iyer, S.H., 1976. In Situ Tests for Ice Strength Measurements. J. of Ocean Eng., Vol. 3, pp. 329-342.

Korzhavin, K.N., 1962. Action of Ice on Engineering Structures. U.S.S.R. Academy Science Siberian Branch, Draft Translated by Cold Regions Res. Eng. Lab., 1971, USA.

Laeroix, R.L. and Pliskin, L., 1970. Prestressed Concrete Gravity Platform for Deep Water. Proc. Offshore Tech. Conf., OTC 1888, Houston, Texas pp. 605-612.

Lauritzsen, P. and Schjetne, K. 1976. Stability Calculations for Offshore Gravity Structures. Proc. Offshore Tech. Conf. Houston, Texas. 101.1 pp. 75 - 82.

Liesemer, S.V., 1980. Offshore Development Concepts - Sable Island and Hibernia. Proc. Ninth Annual Environmental Workshop on Offshore Hydrocarbon Development, Fairmont, Hot Springs, British Columbia, May.

Lunne, T. and St. John, 1979. The use of cone penetration tests to compute Penetration Resistance of Steel Skirts Underneath North Sea Gravity Platforms. Marine Geotechnology. Vol. 3. pp: 369-421.

Maattanen, M., 1975. Ice Forces and Vibrational Behaviour of Bottom Founded Steel Lighthouses. III IAHR/AIRH Int. Symp. on Ice Problems, Hanover, New Hampshire, U.S.A., 11 pp.

Maattanen, M., 1977a. Ice Force Measurements at the Gulf of Bothnia by the Instrumented Kemi-I Lighthouse. Proc. POAC, Memorial University of Newfoundland, St. John's, Nfld., September 26-30., pp: 730-740.

Maattanen, 1977b. Stability of Self-Excited Ice-Induced Structural Vibrations. Proc. POAC '77, Memorial University of Newfoundland, St. John's, Nfld., September 26-30., pp. 684-694.

Maattanen, M., 1978. On Conditions for the Rise of Self-Excited Ice-Induced Autonomous Oscillations in Slender Marine Pile Structures, Research Report No. 25. Department of Mechanical Engineering, University of Olu, Finland.

Makinen, E., Lathi, A. and Rimppi, M., 1975. Influence of Friction on Ice Resistance. Search for Low Friction Surfaces. Proc. Symp. on Ice Tech., April, Montreal, Canada, 11 pages.

McCamy, R.C. and Fuchs, R.A., 1954. Wave Forces in Piles: A Diffraction Theory. Tech. Memo No. 6, Beach Erosion Board. Washington.

Mills, R.G., 1972. Design and Installation of the Atlantic Oceanographic Laboratory Stable Platform. Technical Report Series / BI-R-72-4 / July, Bedford Institute of Oceanography, Dartmouth, N.S.

Moan, T., Haver, S. and Vinje, T., 1975. Stochastic Dynamic Response Analysis of Offshore Platforms with Particular Reference to Gravity-Type Platforms. Proc. Offshore Tech. Conf. OTC 2407, Houston, Texas, pp. 707-720.

Morgenstern, N.R. and Price, V.E. 1965. The Analysis of the Stability of General Ship Surfaces, Geotechnique, Vol. 15. pp. 79-93.

Neill, C.R., 1970, Ice Pressure on Bridge Piers in Alberta. IAHR Symposium Ice and its Action on Hydraulic Structures, Reykjavik, Iceland. pp. 1-8.

Neill, C.R., 1972, Force Fluctuations during Ice-Floe Impact on Piers. Ice Symposium, Leningrad, U.S.S.R.

Neill, C.R., 1976, Dynamic Ice Force on Piers and Piles. An Assessment of Design Guidelines in the Light of Recent Research. Canadian Civ. Eng. Vol. 3, No. 2., pp. 305-341.

Osheima, Nauta, Yashima, and Tabuchi: 1980. Model and Field Experiments for Development of Ice Resistant Offshore Structure. Proc. Offshore Technology Conf. OTC-3885, Vol. 4, pp. 307-314, Texas, USA.

Parmerter, R. and Coon, M.D., 1972, Model of Pressure Ridge Formation in Sea Ice. J. of Geophysical Research, Vol. 77, No. 33, November, pp. 6565-6576.

Pearce, J.C. and Strickland, G.E., 1979. Ice Forces on Conical Structures. Proc. Offshore Tech. Conf. OTC 3635, Houston, Texas., pp. 2407-2416.

Peyton, H.R., 1966a. Sea Ice in Cook Inlet. ASCE Pacific Northwest Council.

Peyton, H.R., 1966b. Sea Ice Forces. Proc. Conf. on Ice Pressure Against Structures. Laval University, Quebec, Nov. National Research Council of Canada, Ottawa. NRC No. 9851, 117-124 (1968).

Peyton, H.R., 1966c. Sea Ice Strength, Report No. UAGR-182, Final Report prepared for Department of Navy, Office of Naval Research by Geophysical Institute, Univ. of Alaska, College 273 pp.

Peyton, H.R., 1968. Ice and Marine Structures, Part I - The Magnitude of Ice Forces Involved in Design Ocean Industry 3: 40-44. Also Part II, 3: 59-65 and Part III, Vol. 3, No. 12, 51-58.

Ralston, T.D., 1977, Ice force design conditions for Conical Structures. Proc. POAC '77, Memorial University of Newfoundland, St. John's, Nfld., Sept. 26-30., pp: 741-752.

Ralston, T.D., 1979. Plastic Limit Analysis of Sheet Ice Loads on Conical Structures. IUTAM Symposium on the Physics and Mechanics of Ice, Copenhagen, Norway, August 6-10, 20 pp.

Reddy, D.V., Cheema, P.S. and Swamidas, A.S.J., 1975, Ice Force Response Spectrum Model Analysis of Offshore Towers. Proc. POAC Univ. Alaska, pp. 304-314.

Roy, F.E., Der, C. and Gibson, W., 1978. Stress due to Wave Forces in an Offshore Research Platform. Proc. Offshore Tech. Conf. OTC 3108, Houston, Texas., pp: 493-502.

Schjetne, K., Anderson, K.H., Lauritzsen and Hansteen 1979. Foundation Engineering for offshore Gravity Structures. Marine Geotechnology. Vol. 3. pp. 369-421.

Stenning, D.G. and Schumann, C.G., 1979. Arctic Offshore Deep Water Ice Structure Interactions. Proc. Offshore Tech. Conf. , OTC 3630, Houston, Texas pp. 2357-2365.

Swamidas, A.S.J., Reddy, D.V. and Purcell, G., 1976, Artificially Generated Ice Force Records. J. of Glaciology, Cambridge, England, pp. 265-283.

Utt, M.E., Durning, P.J., Duthweiler, R.C. and Engle, D.D., 1976, Estimation of the Foundation Condition of a Fixed Platform by Measurement of Dyanmic Response. Proc. Offshore Tech. Conf. OTC 2625, pp. 65-79.

Vaughan, P., 1976. Stability Analysis of Large Gravity Structures. Proc. Conf., Behaviour of Offshore Structure Trondheim, Norway.

Watt, B.J., 1977. Basic Structural Systems - A Review of Their Design and Analysis Requirements. Numerical Method in Offshore Engineering. Ed. Zienkiewicz, Lewis and Stagg. John Wiley, pp. 1 - 41.

Wittman, W. and Schule, J.J., 1966. Comments on the Mass Budget of Arctic Pack Ice. Proc. Symposium on the Arctic Heat Budget and Atmospheric Circulation, The RAND Corp., Santa Monica, California, pp. 215-246.

Wojnarowski, M.E., Stainsen, S.G. and Reddy, N., 1977. Structural Integrity Evaluations of a Fixed Platform using Vibration Criteria. Proc. Offshore Tech. Conf. OTC 2909, Houston, Texas., pp. 247-256.